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TUNNEL BORING MACHINE TECHNOLOGY FOR A DEEPLY BASED MISSILE SYSTEM

Volume I of II
Application Feasibility

Part 2 of 2

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Final Report

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AIR FORCE WEAPONS LABORATORY
Air Force Systems Command
Kirtland Air Force Base, NM 87117

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Tunneling
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

Technical feasibility and cost studies were made for a deep-based missile (DBM) tunnel system (Mesa concept) by means of tunnel boring machines (TBMs), along with designing an egress machine for post-attack tunneling through approximately 2,500 feet of probably unstable rock to the rubble zone.

Currently, available designs of TBMs are readily adaptable for the conventional excavation in geologic environment considered suitable for DBM siting. Most, if not all, of the tunnel sections in the rocks of anticipated structure

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and strength will require support varying from simple rock bolting to concrete segments.

Current (1979) costs for similar tunnels (Chicago) vary from $600 to $800 per linear foot of tunnel, while the estimated costs for the DBM tunnels average as high as $1,600 per foot because of the greater depths, weaker rock, longer tunnels, possible remoteness geographically, and other related factors.

Two concepts for egress machines have been proposed by the Robbins and Jarva companies.

The details of use of geotechnical data are given in Appendix A and were qualitatively for estimates of support requirements and costs. The only calculation that could be made based upon available data was the assumption that squeezing ground would occur if the stress concentration at the ribs of the tunnels exceeds the unconfined compressive strength. Average conditions assumed for the tunnel calculations in the COSTUN program automatically include the effects of rock quality designation (RQD), etc.

APPENDIX A

ENGINEERING FACTORS IN TUNNELING TECHNOLOGY

Many of the basic geotechnical and engineering principles for tunneling in soil and rock are described in the publication by Golder and MacLaren (Ref. A1). The basic information described is employed as input to computer programs, as a means of determination of support requirements, penetration rates, advance rates, and other variable factors which must be carefully evaluated for the determination of tunneling costs.

In short, these principles are the basis for making dependable engineering judgments as represented by the present state of the art and are abstracted here in detail to show that basis for recommendations made in this study.

Geotechnical Consideration

Subsurface conditions have a very critical influence upon the design, excavation, and construction of tunnels, and hence, have an important effect upon both direct and indirect costs. The uncertainties involved in predicting underground conditions constitute the greatest single risk of tunnel construction and considerable research effort has been devoted to the development of new methods of determining geologic structure, pertinent rock properties, ground water level, and the related geological factors prior to excavation. While much progress has been made in the techniques of prediction of underground geology, diamond drilling, geologic mapping, and seismic evaluation remain the most usable tools for site evaluation.

A1. Golder Associates & J.F. MacLaren, Ltd., "Tunneling Technology - An Appraisal of the State of the Art for Application to Transit Systems," Ontario Ministry of Transportation and Communications, May 1976.

Site investigation and geologic evaluation of deep base missile sites will also be one of the major considerations in this project. The cost of diamond drilling and geotechnical evaluation must be balanced against the benefits, and bids for construction of the complex will increase as some function of the risks involved.

For a presently assumed hypothetical model site for 480 km of tunnel, the geologic factors can be evaluated only in terms of predicted percentages of the types of structure that will be encountered. Hence, the costs, rates of excavation, and construction, together with related factors, may be based upon the assumption of the percentage of (1) the worst conditions, (2) medium conditions, and (3) the most favorable conditions. This will give a range of costs to be expected until an actual site is selected. Geologic information can then be employed to make more firm predictions, to reduce risks, and to offer a base for contractual bidding.

The problem of determining the effect of rock properties upon tunneling operations and construction has been approached in several different ways. It is necessary to evaluate rock mass properties, as described below, one of the purposes of which is to determine the tunnel support required, as well as the method and cost of excavation.

A definitive description of geotechnical factors is given in Reference A1. Three classifications of rock are used:

1. According to origin: igneous, sedimentary, and metamorphic
2. According to compressive strength:

Rock Class	Compressive Strength
Very low strength	125 - 500 lb/in^2
Low strength	500 - 2,000 lb/in^2
Medium strength	2,000 - 8,000 lb/in^2
High strength	8,000 - 32,000 lb/in^2
Very high strength	greater than 32,000 lb/in^2

3. According to spacing of discontinuities:

Spacing Class	Spacing
Very wide	Greater than 10 ft
Wide	3 - 10 ft
Moderately wide	1 - 3 ft
Close	2 in. - 1 ft
Very close	Less than 2 in.

These are further qualified by a classification of rock related to tunnel support by Terzaghi (Ref. A2):

"Intact rock contains neither joints nor hair cracks, hence, if it breaks, it breaks across sound rock. On account of the injury to the rock due to blasting, spalls may drop off the roof several hours or days after blasting. This is known as spalling condition. Hard, intact rocks may also be encountered in the popping condition involving the spontaneous and violent detachment of rock slabs from sides or roof.

"Stratified rock consists of individual strata with little or no resistance against separation along the boundaries between strata. The strata may or may not be weakened by transverse joints. In such rock, the spalling condition is quite common.

"Moderately jointed rock contains joints and hair cracks, but the blocks between joints are locally grown together or so intimately interlocked that vertical walls do not require lateral support. In rocks of this type, both the spalling and the popping condition may be encountered.

"Blocky and seamy rock consists of chemically intact or almost intact rock fragments which are entirely separated from each other and imperfectly interlocked. In such rock, vertical walls may require support.

"Crushed but chemically intact rock has the character of a crusher run. If most or all of the fragments are as small as fine sand grains and no recementation has taken place, crushed rock below the water table exhibits the properties of a water-bearing sand.

"Squeezing rock slowly advances into the tunnel without perceptible volume increase. Prerequisite for squeeze is a high percentage of microscopic and sub-microscopic particles of micaceous minerals or of clay minerals with a low swelling capacity.

A2. Terzaghi, K., 1946, "An Introduction to Tunnel Geology," Rock Tunneling with Steel Supports, by R.V. Proctor and T.L. White, The Commercial Shearing and Stamping Co., Youngstown, Ohio, USA.

"Swelling rock advances into the tunnel chiefly on account of expansion. The capacity to swell seems to be limited to those rocks which contain clay minerals, such as montmorillonite, with a high swelling capacity.

"In practice, there are no sharp boundaries between these rock categories and the properties of the rocks indicated by each one of these terms can vary between wide limits."

These rock classes relate to tunnel support requirements in terms of load and stand-up time (time after excavation until first movement occurs).

Lauffer (Ref. A3) describes a similar classification system:

Class A: Stable rock - corresponds to intact rock as identified by Terzaghi.

Class B: Rock unstable after a long time - corresponds to massive moderately jointed rock and to stratified or schistose rock.

Class C: Unstable rock after a short time - corresponds to moderately blocky and seamy rock.

Class D: Broken rock - corresponds to very blocky and seamy rock.

Class E: Very broken rock - corresponds to completely crushed rock.

Class F: Squeezing rock.

Class G: Heavy squeezing rock.

Lauffer further proposed a quantitative correlation between the above rock classes and the stand-up time of the wall rock in tunnel openings of various sizes (Figure A1).

Since 1965, factors other than general classifications, such as those above, and rock properties have been employed to improve the predictability of rock stability and of factors affecting excavation. One of the first of these is the RQD (Rock Quality Designation) which is based on particle size or fracture spacing. These factors can be measured and used to determine:

A3. Lauffer, H., 1950, "Gebirgsklassifizierung fur den Stollenbau," Geologie und Bauwesen, 24, H.1., Vienna, Austria.

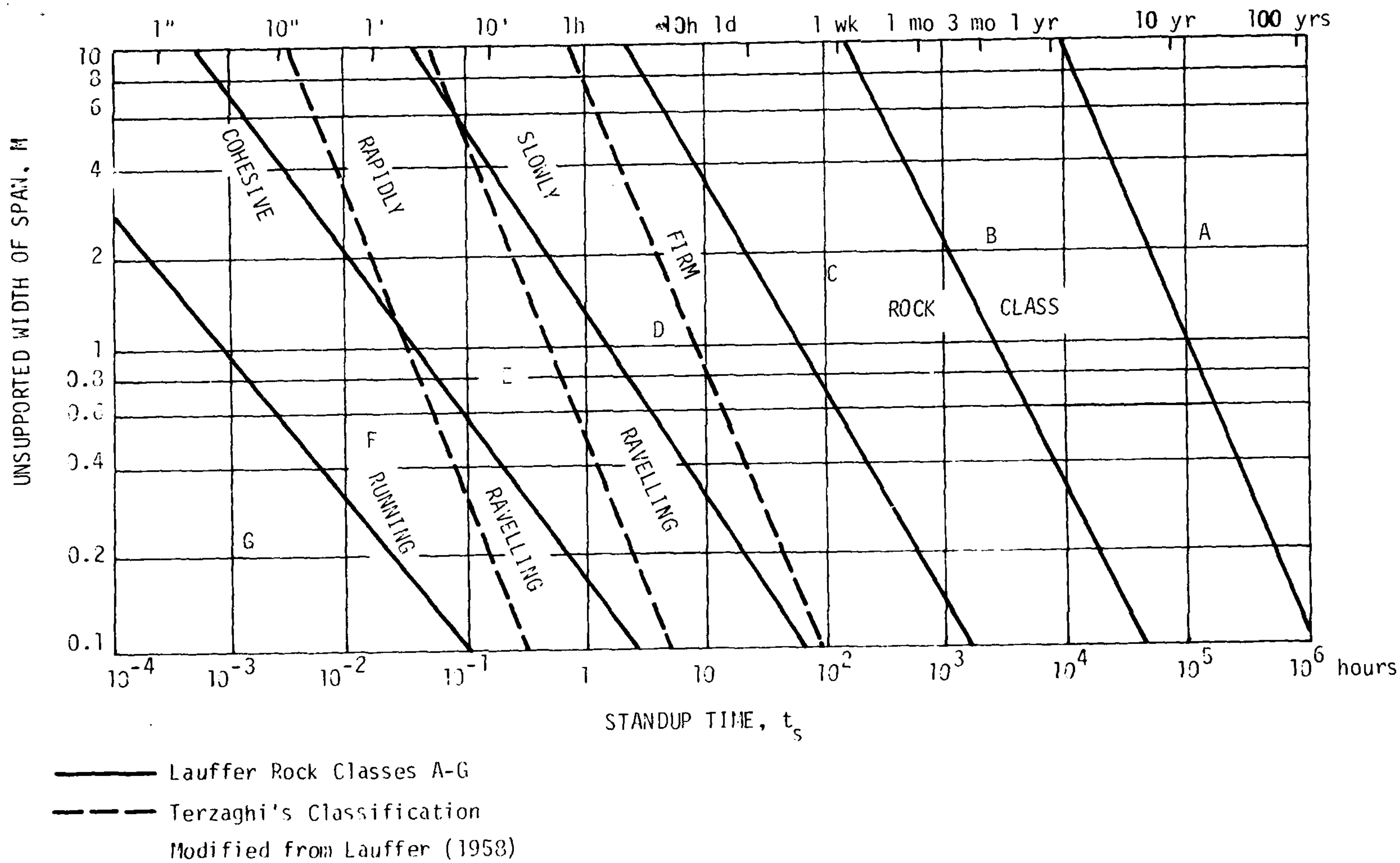

FIGURE A1 - Standup Time as a Function of Rock Class and Unsupported Width of Tunnel Roof (Ref. A4)

1. The behavior of the rock mass as a function of the type and spacing of discontinuities, as compared to the size of the tunnel.

2. The strength, deformability, permeability, and other properties of the rock.

3. The overall behavior of the rock mass around the tunnel as affected by the above properties and the larger scale geologic structure.

The RQD can serve to define the limits of applicability of design theories such as rock mechanics and continuum mechanics, the factors controling ground behavior, and to establish broad behavioristic patterns. Beyond gravel size (5 mm), the RQD is preferred, which is defined as the sum of the lengths of recovered core pieces which are 4 in. or longer, divided by the total length of the core:

RQD	Rock Quality
0 - 25%	Very poor
25 - 50%	Poor
50 - 70%	Fair
75 - 90%	Good
90 - 100%	Excellent

Bieniawski (Ref. A4) proposed a classification system (Council for Scientific & Industrial Research - CSIR) which includes the RQD and other factors:

1. Rock Quality Designation (RQD)
2. State of weathering
3. Uniaxial compressive strength of intact rock
4. Spacing of joints and bedding
5. Strike and dip orientations
6. Separation of joints

A4. Bieniawski, Z.T., 1973, "Engineering Classification of Jointed Rock Masses," The Civil Engineer in South Africa, South African Institution of Civil Engineers, Transvaal, South Africa, pp. 335-343.

7. Continuity of joints

8. Groundwater flow

Weathering

Unweathered rock shows no signs of visible weathering, rock is fresh, and few discontinuities may show slight staining. Slightly weathered rock shows penetrative weathering in open discontinuity surface but only slight weathering of rock material. Discontinuities are discolored extending into rock up to 10 mm.

Moderately weathered rock exhibits slight discoloration extending through the greater part of the rock mass. The rock material is not friable (except for poorly cemented sedimentary rocks).

In highly weathered rock, the weathering extends throughout the rock mass, and the rock material is partly friable. Rock has no lustre, and all minerals except quartz are discolored. Rock can be excavated with a pick.

Completely weathered rock is totally discolored, decomposed, and friable with only fragments of the rock texture and structure preserved; it has the external appearance of a soil.

Uniaxial Compressive Strength of Intact Rock

The classification of intact rock proposed by Deere and Miller (Ref. A5) has been accepted as convenient and realistic (Table A1).

Spacing of Joints

The term joint is used to mean all discontinuities including joints, faults, bedding planes, and other surfaces (Table A2).

A5. Deere and Miller, 1966, University of Illinois, Champaign-Urbana, Illinois.

TABLE A1

Deere and Miller's Classification of Intact Rock Strength

Description	Uniaxial Compressive Strength lbf/in^2	Uniaxial Compressive Strength Kgf/cm^2	Uniaxial Compressive Strength MPa	Examples of Rock Types
Very low strength	150- 3500	10- 250	1- 25	Chalk, rocksalt
Low Strength	3500- 7500	250- 500	25- 50	Coal, siltstone, schist
Medium Strength	7500-15000	500-1000	50-100	Sandstone, slate, shale
High Strength	15000-30000	1000-2000	100-200	Marble, granite, gneiss
Very High Strength	>30000	>2000	>200	Quartzite, dolerite, gabbro, basalt

TABLE A2

Deere's Classification for Joint Spacing

Description	Spacing of Joints	Rock Mass Grading
Very Wide	>3m	>10 ft - Solid
Wide	1m to 3m	3 ft to 10 ft - Massive
Moderately Close	0.3m to 1m	1 ft to 3 ft - Blocky/Seamy
Close	50mm to 300mm	2 in. to 1 ft - Fractured
Very Close	< 50mm	< 2 in. - Crushed and shattered

Strike and Dip Orientations

Based upon observations of Wickham, Tiedeman, and Skinner (Ref. A6), Bieniawski suggests a qualitative assessment such as favorable or unfavorable should be used in preference to a quantitative classification of the effects of discontinuity orientation and inclination.

Separation of Joints, Continuity of Joints and Groundwater

These three factors have a significant influence upon the behavior of the rock mass and are taken into account in a qualitative manner.

The eight parameters discussed above have been incorporated into a classification (Table A3) in which each parameter is graded into five classes ranging from very good to very poor.

Each parameter does not necessarily contribute equally to the behavior of a rock mass, and one may have a rock mass in which the different parameters fall into different classes. For example, an RQD of 80, which places the rock mass in class 2, may be associated with a heavy groundwater inflow which would place the rock mass in class 4 or 5.

To overcome this difficulty, an Importance Rating weighting factor is used to allocate points for each class number of each parameter, and the overall class of the rock mass is then determined by adding up the total (Table A4).

In the example given, the class was determined from Table A3 and the individual scores from Table A4-A. Table A4-B shows that the total score of 64 places this rock mass in class 3, which is described as fair.

A6. Wickham, G.E., H.R. Tiedeman, and E.H. Skinner, "Support Determination Based Geologic Predictions," Proceedings 1st North American Rapid Excavation & Tunneling Conference, AIME, New York, N.Y., USA, 1972.

TABLE A3

CSIR Geomechanics Classification for Jointed Rock Masses

Item	Class No. & Description	1 Very Good	2 Good	3 Fair	4 Poor	5 Very Poor
1	Rock Quality RQD %	90-100	75-90	50-75	25-50	<25
2	Weathering	Unweathered	Slightly Weathered	Moderately Weathered	Highly Weathered	Completely weathered
3	Intact Rock Strength, MPa	>200	100-200	50-100	25-50	<25
4	Joint Spacing	>3m	1m-3m	0.3m-1m	50mm-300mm	<50mm
5	Separation of Joints	<0.1mm	<0.1mm	0.1mm-1.0mm	1mm-5mm	>5mm
6	Continuity of Joints	Not Continuous	Not Continuous	Continuous, No Gouge	Continuous, With Gouge	Continuous, With Gouge
7	Groundwater Flow per 10m	None	None	Slight <25.5 lit/min	Moderate 25-125 lit/min	Heavy >125 lit/min
8	Strike & Dip Orientations	Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable

TABLE A4

Importance Ratings

A. Individual Ratings for Classification Parameters for Underground Excavations

Underground Excavations						
		Class Number				
Item	Parameter	1	2	3	4	5
1	Rock Quality - RQD	16	14	12	7	3
2	Weathering	9	7	5	3	1
3	Intact Rock Strength	10	5	2	1	0
4	Spacing of Joints	30	25	20	10	5
5	Separation of Joints	5	5	4	3	1
6	Continuity of Joints	5	5	3	0	0
7	Groundwater	10	10	8	5	2
8	Strike & Dip Orientations	15	13	10	5	3

B. Total Ratings for Rock Mass Classes

Class Number	1	2	3	4	5
Description of Class	Very Good	Good	Fair	Poor	Very Poor
Total Rating	90-100	70-90	50-70	25-50	<25

Practical Example Using CSIR Geomechanics Classification

Consider the example of a weathered granitic rock mass in which a tunnel is to be driven. The classification of this rock mass would be carried as follows:

Item	Description	Class	Score
1. RQD	70%	3	12
2. Weathering	Moderate	3	5
3. Intact Strength	150 MPa	2	5
4. Joint Spacing	2m	2	25
5. Joint Separation	0.5mm	3	4
6. Joint Continuity	Cont., No gouge	3	3
7. Groundwater	Moderabe	4	5
8. Strike/Dip	Unfavorable	4	5
		TOTAL SCORE	64

The above description of the CSIR classification system was published with permission from, "Underground Excavation Engineering," by E. Hoek (Ref. A1).

Barton, Lien, and Lunde (Ref. A7) of the Norwegian Geotechnical Institute (NGI) also devised an index for the determination of the tunneling quality of a rock mass for support requirements. This employs the RQD with multiplying and dividing coefficients to account for the influence of joint set (J_s), joint roughness (J_r), joint alteration (J_a), and joint water (J_w), and also a stress reduction factor (SRF). The resulting quality index (Q) is given by $Q = (RQD/J_n)\ (J_r/J_a)\ (J_w/SRF)$. The values for the J's and the SRF (Tables A5, A6, & A7) are from Reference A8.

"The factor (J_w/SRF) involves two stress parameters. SRF is a measure of: (1) load caused by loosening of rock in an excavation through shear zones and clay-bearing rock, (2) reduction of rock stress load in competent rocks, and (3) squeezing loads in plastic incompetent rocks. J_w is a measure of water pressure, which reduces the shear strength of joints due to a reduction in normal stress. Water may also cause softening and possible outwash of clay-filled joints. It has proved impossible to combine these two parameters in terms of interblock effective normal stress, because paradoxically a high value of effective normal stress may sometimes signify less stable conditions than a low value, despite the higher shear strength. The quotient (J_w/SRF) is a complicated empirical factor describing the 'active stresses'."

A7. Barton, N., R. Lien, and J. Lunde, June 1974, "Analysis of Rock Mass Quality and Support Practice in Tunneling and a Guide to Estimating Support Requirements," Report No. 54206, Norwegian Geotechnical Institute; also "Engineering Classification of Rock Masses for the Design of Tunnel Support," Rock Mechanics, Vol. 6, New York, N.Y., USA.

TABLE A5

Descriptions and Ratings for the Parameters RQD, J_n and J_r

1.	Rock Quality Designation	(RQD)	Note 1:
A.	Very Poor	0- 25	(a) Where RQD is reported or measured as ≤ 10 (including 0) a nominal value of 10 is used to evaluate Q in Eq. (1)
B.	Poor	25- 50	
C.	Fair	50- 75	
D.	Good	75- 90	
E.	Excellent	90-100	
2.	Joint Set Number	(J_n)	(b) RQD intervals of 5, i.e. 100, 95, 90, etc. are sufficiently accurate
A.	Massive, no or few joints	0.5-1.0	Note 2:
B.	One joint set	2	(a) For intersections use (3.0 x J_n)
C.	One joint set plus random	3	(b) For portals use (2.0 x J_n)
D.	Two joint sets	4	
E.	Two joint sets plus random	6	
F.	Three joint sets	9	
G.	Three joint sets plus random	12	
H.	Four or more joint sets, random, heavily jointed, "sugar cube", etc.		
J.	Crushed rock, earthlike		
3.	Joint Roughness Number	(J_r)	Note 3:
	(a) Rock wall contact and (b) Rock wall contact before 10 cms shear		(a) Add 1.0 if the mean spacing of the relevant joint set is greater than 3m
A.	Discontinuous joints	4	(b) J_r = 0.5 can be used for planar slickensided joints having lineations, provided the lineations are favorably oriented.
B.	Rough or irregular, undulating	3	
C.	Smooth, undulating	2	
D.	Slickensided, undulating	1.5	
E.	Rough or irregular, planar	1.5	
F.	Smooth, planar	1.0	
G.	Slickensided, planar	0.5	
	(c) No rock wall contact when sheared		
H.	Zone containing clay minerals thick enough to prevent rock wall contact	1.0 (nominal)	
I.	Sandy, gravelly or crushed zone thick enough to prevent wall contact	1.0 (nominal)	

TABLE A6

Descriptions and Ratings for the Parameters J_a and J_w

4.	Joint Alteration Number	(J_a)	τ_r (approx.)
	(a) Rock wall contact		
A.	Tightly healed, hard, non-softening, impermeable filling i.e. quartz or epidote	0.75	(-)
B.	Unaltered joint walls, surface staining only	1.0	(25°-35°)
C.	Slightly altered joint walls. Non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.		
D.	Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20°-25°)
E.	Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also Chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2 mm or less in thickness)	4.0	(8°-16°)
	(b) Rock wall contact before 10 cms shear		
F.	Sandy particles, clay-free disintegrated rock etc.	4.0	(25°-30°)
G.	Strongly over-consolidated, non-softening clay mineral fillings. (Continuous, 5mm in thickness)	6.0	(16°-24°)
H.	Medium or low over-consolidation, softening, clay mineral fillings. (Continuous, 5mm in thickness)	8.0	(12°-16°)
J.	Swelling clay filling, i.e. montmorillonite. (Continuous, 5mm in thickness. Value of J_a depends on percent of swelling clay-size particles, and access to water etc.	8.0-12.0	(6°-12°)
	(c) No rock wall contact when sheared		
K,L, M.	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6.0,8.0	(6°-24°)
N.	Zones or bands of silty- or sand-clay, small clay fraction (non-softening)	5.0	
O,P, R.	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	10.0,13.0 or 13.0-20.0	(6°-24°)

Note: (i) Values of $(\tau)_r$ are intended as an approximate guide to the mineralogical properties of the alteration products, if present.

5.	Joint Water Reduction Factor	(J_w)	Approx. water pressure (kg/cm^2)
A.	Dry excavations or minor inflow, i.e. <5l/min locally	1.0	<1
B.	Medium inflow c. pressure occasional outwash of joint fillings	0.66	1.0-2.5
C.	Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5-10.0
D.	Large inflow or high pressure, considerable outwash of joint fillings	0.33	2.5-10.0
E.	Exceptionally high inflow or water pressure at blasting, decaying with time	0.2-0.1	>10.0

TABLE A6 (Cont'd.)

5. Joint Water Reduction Factor	(J_w)	Approx. water pressure (kg/cm^2)
F. Exceptionally high inflow or water pressure continuing without noticeable decay	0.1-0.05	>10.0
Note: (i) Factors C to F are crude estimates. Increase J_w if drainage measures are installed.		
(ii) Special problems caused by ice formation are not considered.		

TABLE A7

Descriptions and Ratings for the Parameter SRF

6.	Stress Reduction Factor			(SRF)
	(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.			
A.	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0
B.	Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation $\leq$ 50 m)			5.0
C.	Single weakness zones containing clay, or chemically disintegrated rock (depth of excavation 50 m)			2.5
D.	Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)			7.5
E.	Single shear zones in competent rock (clay free) (depth of excavation $\leq$ 50 m)			5.0
F.	Single shear zones in competent rock (clay free) (depth of excavation 50 m)			2.5
G.	Loose open joints, heavily jointed or "sugar cube" etc. (any depth)			5.0
		σ_c/σ_1	σ_t/σ_1	
H.	Low stress, near surface	>200	>13	2.5
J.	Medium stress	200-10	13-0.66	1.0
K.	High stress, very tight structure. (Usually favorable to stability, may be unfavorable to wall stability)	10-5	0.66-0.33	0.5-2.0
L.	Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M.	Heavy rock burst (massive rock)	<2.5	<0.16	10-20
	(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures.			
N.	Mild squeezing rock pressure			5-10
O.	Heavy squeezing rock pressure			10-20
	(d) Swelling rock; chemical swelling activity depending on presence of water.			
P.	Mild swelling rock pressure			5-10
R.	Heavy swelling rock pressure			10-15

Note: (i) Reduce these values of SRF by 25-50% if the relevant shear zones only influence but do not intersect the excavation.

(ii) For strongly anisotropic stress field (if measured): when $5 \leq \sigma_1/\sigma_3 = 10$, reduce σ_c and σ_t to 0.8 σ_c and 0.8 σ_t; when $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to 0.6 σ_c and 0.6 σ_t where: σ_c = unconfined compression strength, σ_t = tensile strength (point load), σ_1 and σ_3 = major and minor principal stresses.

(iii) Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

The rock tunnelling quality Q is then considered as a function of three parameters which are crude measures of:

1. Block size (RQD/J_n)
2. Interblock shear strength (J_r/J_a)
3. Active stress (J_w/SRF)

Joint orientation was not found to be the important general parameter as might be expected because the orientation of the excavation can be adjusted to avoid the maximum effect of unfavorably oriented major joints. This choice is not usually available for tunnels. The parameters, J_n, J_r, and J_a may play a more important general role than orientation because the number of joint sets determines the freedom for block movement.

The information contained in Tables A5 to A7 may be easily used to determine the NGI Tunnelling Quality Index. The process of determining the various factors required for computation of Q concentrates the attention of the user on a number of important practical questions which might be ignored during a site investigation. The qualitative evaluation of characteristics of the rock mass which is required for this process may be almost as important as the calculated value of Q.

The first factor (RQD/J_n) represents the structure of the rock mass, is a rough measure of the block or particle size varying between two extreme values (100/1.5 and 10/20) differing by a factor of 400, giving extreme particle sizes of 200 to 0.5 cm, which are crude but fairly realistic approximations. The largest blocks may be several times the upper limit and the smallest fragments less the lower limit.

The second factor (J_r/J_a) represents the roughness and frictional characteristics of the joint walls or filling materials. It is weighted in favor of rough, unaltered joints which are in direct contact. Such surfaces will be close to peak strength, they will tend to dilate strongly when

sheared, and will, therefore, be favorable to tunnel stability. Thin clay mineral coatings and fillings in the joints reduce the strength significantly. Nevertheless, joint surface contact after small shear displacements have occurred may be a very important factor in preserving the stability of the opening. If no joint surface contact exists, the conditions are extremely unfavorable. The friction angles given in Table All are a little less than the residual strength values for most clays.

Rock Structure Rating (RSR). Jacobs and Associates (Ref. A8) describe a Ground Support Prediction Model for a method for evaluating and rating on a numerical scale the competency of a rock structure to determine the need for structural support. The model has evolved from study of ground support installations and geological information for 53 tunnel projects to provide an index or Rock Structure Rating (RSR).

The need for ground support is influenced by geological and construction factors. Geological factors include the rock-joint patterns, the dip and strike, discontinuities, faults, shears and folds, groundwater, rock material properties, and the degree of weathering or alteration. Construction factors are the size of openings, direction of drive, and method of excavation. The prediction of ground support requirements takes these factors into account but is influenced by attitude and experience of the predictor.

The RSR is the sum of three parameters: Parameter A represents geologic factors such as the effects of rock type (igneous, metamorphic, sedimentary), strength (hard to soft), and the geological structures (folds

A8. Jacobs and Associates, January 1974, "Ground Support Prediction Model (RSR Concept)," Technical Report No. 125, for U.S. Bureau of Mines, Washington, D.C., USA, Contract No. H0220075.

and faults); Parameter B represents the joint pattern, joint spacing, thickness, and the strike and dip with respect to the direction of tunnel drive; and, Parameter C represents the groundwater and joint conditions.

Each has been evaluated based upon past experience and weighted numerical values assigned to reflect their effect on the overall support requirement. Tables of the values of A, B, and C prepared in 1972 were revised following a critical study by tunnelling specialists and further checks against observations on 25% projects (Tables A8, A9, & A10).

The rock structure rating of the rock mass at a tunnelling site is defined as the sum of the values of A, B, and C. This RSR will vary from a low value of 19 for the worst rock conditions to a maximum of 100 for ideal conditions.

Site Investigations. A successful geotechnical investigation must be carried out to provide the information necessary for the following:

1. Conceptual design decisions, for example, on the depth, location, alignment, and geometry of the tunnel opening.
2. Detailed design, the selection of the construction method, and of the support system.
3. Detailed prediction and evaluation of construction problems.

The investigation is carried out in successive stages, each stage leading to a major design decision, the next stage narrowed in extent and scope, but developed in order to lead to the next design decision.

Initially, the investigation should be concerned with the geology of the area and utilize the classical tools available to the geologist, i.e., geologic maps and reports, geophysical exploration, and borings, with care exercised by the engineer in assessing the validity of the geological maps.

TABLE A8

Rock Structure Rating, Parameter "A", General Area

Geology From Jacobs Associates, 1974 (Ref. A9)				
Basic Rock Type	Hard	Medium	Soft	Decomposed
Igneous	1	2	3	4
Metamorphic	1	2	3	4
Sedimentary	2	3	4	4

Geological Structure

	Massive	Slightly Faulted or Folded	Moderately Faulted or Folded	Intensely Faulted or Folded
Type 1	30	22	15	9
Type 2	27	20	13	8
Type 3	24	18	12	7
Type 4	19	15	10	6

TABLE A9

Rock Structure Rating, Parameter "C", Groundwater, Joint Condition (Ref. A9)

Anticipated Water Inflow (gpm/1000')	Sum of Parameters A + B 13-44 Joint Condition Good	Fair	Poor	45-75 Good	Fair	Poor
None	22	18	12	25	22	18
Slight (<200 gpm)	19	15	9	23	19	14
Moderate (200-1000 gpm)	15	11	7	21	16	12
Heavy (>1000 gpm)	10	8	6	18	14	10

Joint Condition

Good = tight or cemented

Fair = slightly weathered or altered

Poor = severely weathered, altered or open

TABLE A10

Rock Structure Rating, Parameter "B", Joint Pattern, Direction of Drive
(Ref. A9)

	Strike ⊥ to Axis Direction of Drive Both with Dip Dip of Prominent Joints			Against Dip		Strike ∥ to Axis Direction of Drive Both Dip of Prominent Joints		
	Flat	Dipping	Vertical	Dipping	Vertical	Flat	Dipping	Vertical
(1) Very closely jointed	9	11	13	10	12	9	9	7
(2) Closely jointed	13	16	19	15	17	14	14	11
(3) Moderately jointed	23	24	28	19	22	23	23	10
(4) Moderate to blocky	30	32	36	25	28	30	28	24
(5) Blocky to massive	36	38	40	33	35	36	34	23
(6) Massive	40	43	45	37	40	40	38	34

Notes: Flat 0-20°; Dipping 20°-50°; Vertical 50°-90°

"The principal areas of uncertainty in the geological structure relevant to alternative tunnel routes should be carefully considered and checked by borings. Such investigation, covering possible alternative routes over a large area, must be kept relatively general for economic reasons. However, it should be sufficiently detailed in terms of ground characteristics to permit a rough evaluation of the tunnelling methods to be used and of possible tunnel costs. This first stage of the geotechnical investigation will normally lead to the establishment of preliminary tunnel design for all routes considered, to the relative rating of these alternative designs, and to the first selection of what appears as the best route at the stage. In the second stage, the investigation is concentrated on the selected route and is detailed to lead to the evaluation of the ground characteristics necessary for the design of the tunnel excavation and support. Boreholes, laboratory and in situ tests are the main investigation tools. The extent of the program, i.e., the number of boreholes and the type and number of tests, will depend eventually on the variability of the geology. In particular, the investigation should provide, within reasonable expenditure, information on all discontinuities that might have adverse effects on tunnelling.

"At the final design stage, additional geologic information may be required for example, to permit the detailed design of tunnelling machines or the design of new systems of lining. All information gathered during the geotechnical investigation should be compiled and made available to the contractor to allow reasonable bidding with a minimum of major uncertainties. This information will give the contractor the opportunity to investigate and propose innovative approaches to the tunnel design and construction. This latter aspect is particularly important in view of the fact that most of the major improvements in tunnelling technology have resulted from contractor initiatives.

"The geotechnical investigation for any tunnel need not and should not stop at this stage. The construction stage provides a chance to compare 'predicted' to 'actual' geology. This comparison in areas already tunnelled may provide information on the accuracy of predicted geology in future sections.

"The total cost of a geotechnical investigation will depend on a number of factors, such as the location, purpose, and dimensions of the proposed tunnel, the estimated total cost of the project, the difficulties related to local geology, the availability of skilled personnel and equipment, and consideration of time constraints. As discussed by Ash, et al (1974), an increase in the extent and cost of the geotechnical investigation is normally associated with a decrease in the cost of risk and effort involved in the tunnel construction. However, as shown in Figure A2, there is a finite limit to the desirable extent of the geotechnical investigation beyond which little or no overall economy can be achieved. At present, such a limit generally occurs at an investigation cost of 1 to 3 percent of the total cost of the tunnel; however, for specific complex projects, this cost is usually double.

"Suggested steps in a geotechnical investigation for a tunnel are given below.

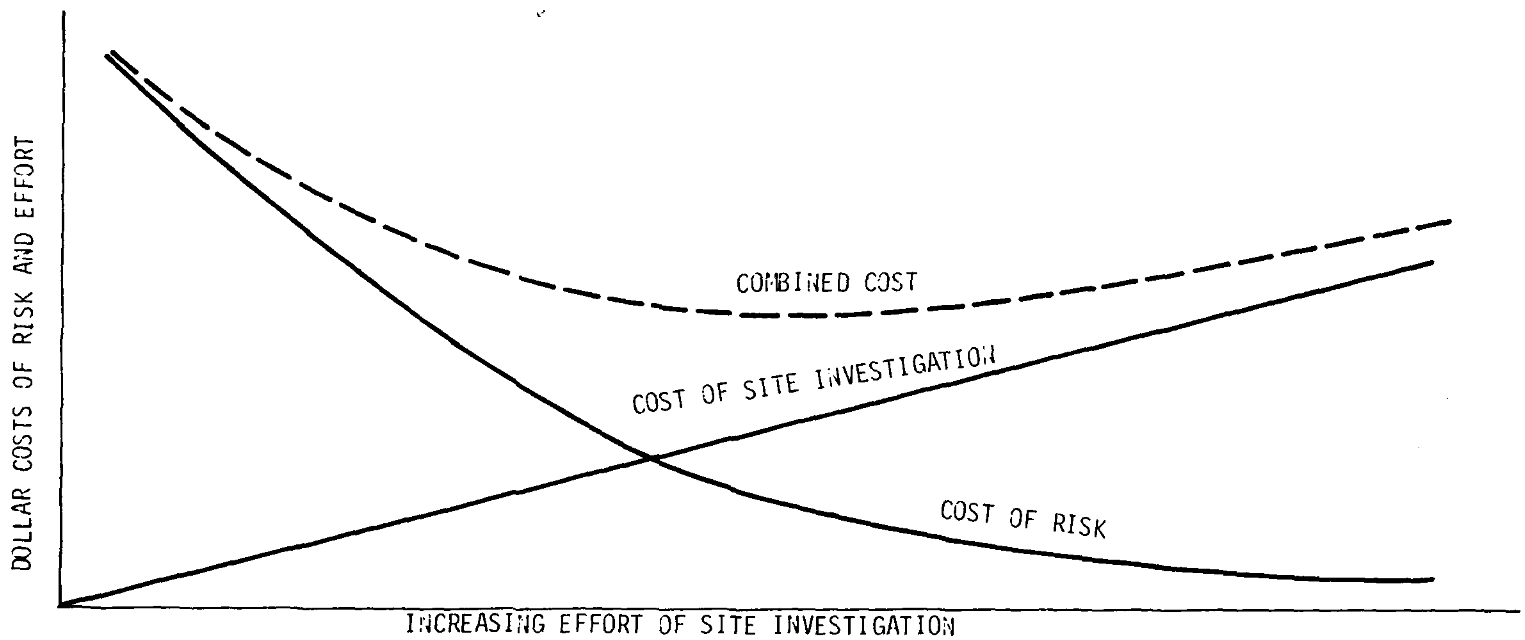

FIGURE A2 - Cost of Site Investigation and Cost of Risk in Tunnel Construction (Ref. A11)

(a) Assemble all the local geotechnical and geological information, and the local geological history and tectonic movements. This information should be used to guide the subsequent investigations and should be in a form which may be used by the contractors at the time of tendering.

(b) Develop a preliminary site investigation procedure which is based upon the following factors:

(i) Principal known areas of uncertainty, such as buried pre-glacial valleys, faults, and beach deposit areas should have at relatively close intervals.
(ii) Between the principal areas of uncertainty, the borings should be put down to provide representative data on typical soil or rock types. This interval may be about 500 feet and should not exceed 750 feet in soft ground, and should not exceed 1,000 feet in rock.

(c) The stratigraphy and geology as determined by the first stage of the investigation should be examined in detail insofar as the design and construction problems associated with tunnelling are concerned.

(d) A second detailed investigation should be designed to eliminate any areas of uncertainty established in the preliminary stage. This work may include:

(i) Detailed intermediate borings to determine the boundaries of the soil and rock types;
(ii) Geophysical and other geological testing methods to develop an inferred stratigraphy between the borings; and,
(iii) Large diameter borings or inspection shafts. The large diameter borings in rock are useful to provide sufficient sample material for testing in terms of rock abrasiveness and cutting, while the test shafts in soft ground provide a means of inspection at the tendering stage and give an idea of the occurrence of boulders.

(e) Throughout all stages of the investigation, piezometric measurements should be made in the borings to determine regional and seasonal variations in groundwater conditions." (Ref. A1).

Recommendations for permanent support are given for 38 categories (Tables A11, A12, A13, & A14). These tables have been designed to give estimates for permanent roof support. Methods for estimating permanent wall support are based on the hypothetical "wall quality" range 1.0 Q to 5.0 Q. An example of calculation procedures is given at the end of the chapter.

TABLE A11

Classification Data for Self-Supporting Tunnels

Support Category	Case No.	Description of Support Used	RQD/J_n	J_r/J_a	J_w/SRF	SPAN/ESR (= D_e)	Q
No. 0	6	none, S (1 app.) for protection from small stones	60/2	2/1	1/1	9/1.6	60
(no support)	8	none	70/2	1/1	1/1	9/1.6	35
	17	sb + S (1 appl) for protection from small stones	100/2	1.5/1	1/1	9/1.6	75
	20	none	70/2	1/1	1/1	9/1.6	35
	21	none	100/1	4/1	0.66/1	13/1.0	266
	27	(near category 13) none	90/3	1/1	1/1	12.5/1.6	30
	29	none	90/2	3/1	1/1	12/5/1.6	135
	35	none	10/3	2/1	1/1	5/1.6	6.7
	36	none	20/2	2/1	1/1	5/1.6	20
	63	(near category 17)B	100/9	1/1	1/2.5	5.9/1.6	4.4
	68	none	100/1/2	5/1	1/1	10/1.0	1000
	70	none	40/2	1.5/1	1/2.5	8/1.6	12
	74	(near category 9) none	100/2	1/1	1/1	12/1.3	16.7
	77	(near category 5)sb (50 bolts per 300m)	100/1	5/1	1/2.5	20/1.3	200
	78	none	90/2	1.5/1	1/2.5	5/1.3	27
	87	none	100/1	4/1	1/1	11.25/1.6	400
	91	none	90/2	1.5/1	1/1	12/1.3	67.5
	96	none	100/1	4/1	?1/2.5	15/1.3	160
	101b	none	75/9	2/3	0.66/1	3.5/1.3	3.7
	112	none	80/2	2/1	1/15	1.2/1.6	5.3
	113	none	100/1	4/1	1/7.5	2.3/1.6	46
	115	(near category 13)B (1.0m)	100/1	4/1	1/20	6.4/1.0	20
	119b	none	100/1	4/1	1/0.5	100/4	800
	119c	none	100/1	4/1	1/0.5	100/5	800
	120a	none	95/9	3/1	1/1	7/1.3	31.6
	120b	none	95/9	3/1	1/0.5	7/1.3	63

TABLE A11(Cont'd.)

Support Category	Case No.	Description of Support Used	RQD/J_N	J_r/J_a	J_w/SRF	SPAN/ESR (= D_e)	Q
	127a	none or sb	100/4	3/1	1/0.5	20/5	75
	127b	none or sb	100/4	3/1	1/0.5	20/3	150
	144	sb, 2 m long	90/4	1/4	1/1	3/1.3	5.6
	150	none	100/4	2/1	0.5/0.5	6.1/1.3	50

Key: S = shotcrete (number of applications in brackets)

B = systematic bolting (mean spacing in brackets)

sb = spot bolting

TABLE A12

Support Measures for Rock Masses of "Exceptional," "Extremely Good," "Very Good," and "Good" Quality (Q range: 1000-10)

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_n	SPAN/ESR(m)	P kg/cm^2 (approx.)	SPAN/ ESR(m)	Type of Support	Note: see Ref. A1
1*	1000-400	---	---	---	<0.01	20- 40	sb (utg)	---
2*	1000-400	---	---	---	<0.01	30- 60	sb (utg)	---
3*	1000-400	---	---	---	<0.01	46- 80	sb (utg)	---
4*	1000-400	---	---	---	<0.01	65-100	sb (utg)	---
5*	400-100	---	---	---	0.05	12- 30	sb (utg)	---
6*	400-100	---	---	---	0.05	19- 45	sb (utg)	---
7*	400-100	---	---	---	0.05	30- 65	sb (utg)	---
8*	400-100	---	---	---	0.05	48- 88	sb (utg)	---
9	100- 40	≥ 20	---	---	0.25	8.5- 19	sb (utg)	---
		< 20	---	---			B (utg) 2.5-3 m	---
10	100- 40	≥ 30	---	---	0.25	14- 30	B (utg 2-3 m)	---
		< 30	---	---			B (utg) 1.5-2 m	---
11*	100- 40	≥ 30	---	---	0.25	24- 48	B (tg) 1.5-2 m	---
		< 30	---	---			B (tg) 1.5-2 m + clm	---
12*	100- 40	≥ 30	---	---	0.25	40- 72	B (tg) 2-3 m	---
		< 30					B (tg) 1.5-2 m + clm	---
13	40- 10	≥ 10	≥ 1.5	---	0.5	5- 14	sb (utg)	I
		≥ 10	< 1.5	---			B (utg) 1.5-2 m	I
		< 10	≥ 1.5	---			B (utg) 1.5-2 m	I
		< 10	< 1.5	---			B (utg) 1.5-2 m + S 2-3 cm	I

TABLE A12 (Cont'd.)

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_n	SPAN/ESR(m)	P kg/cm^2 (approx.)	SPAN/ ESR(m)	Type of Support	Note: see Ref. A1
14	40- 10	$\geq$ 10	---	$\geq$ 15	0.5	9- 23	B (tg) 1.5-2 m + clm	I, II
		< 10	---	$\geq$ 15			B (tg) 1.5-2 m + S (mr) 5-10 cm	I, II
		---	---	< 15			B (utg) 1.5-2 m + clm	I, III
15	40- 10	> 10	---	---	0.5	15- 40	B (tg) 0.5-2 m + clm	I, II, IV
		$\leq$ 10	---	---			B (tg) 1.5-2 m + S (mr) 5-10 cm	I, II, IV
16* See note XII	40- 10	> 15	---	---	0.5	30- 65	B (tg) 1.5-2 m + clm	I, V, VI
		$\leq$ 15	---	---			B (tg) 1.5-2 m + S (mr) 10-15 cm	I, V, VI

*Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth wall blasting and thorough barring-down may remove the need for support. Roughwall blasting may result in the need for single applications of shotcrete, especially where the excavation height is > 25 m. Future case records should differentiate categories 1 to 8.

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted

TABLE A13

Support Measures for Rock Masses of "Fair" and "Poor" Quality (Q range: 10-1)

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_n	SPAN/ESR	P Kg/cm^2 (approx.)	SPAN/ ESR (m)	Type of Support	Note: see Ref. A1
17	10-4	> 30	---	---	1.0	3.5- 9	sb (utg)	I
		≥ 10, ≤ 30	---	---			B (utg) 1-1.5 m	I
		< 10	---	≥ 6 m			B (utg) 1-1.5 m + S 2-3 cm	I
		< 10	---	< 6 m			S 2-3 cm	I
18	10-4	> 5	---	≥ 10 m	1.0	7-15	B (tg) 1-1.5 m + clm	I, III
		> 5	---	< 10 m			B (utg) 1-1.5 m + clm	I
		≤ 5	---	≥ 10 m			B (tg) 1-1.5 m + S 2-3 cm	I, III
		≤ 5	---	< 10 m			B (utg) 1-1.5 m + S 2-3 cm	I
19	10-4	---	---	≥ 20 m	1.0	12-29	B (tg) 1-2 m + S (mr) 10-15 cm	I, II, IV
		---	---	< 20 m			B (tg) 1-1.5 m + S (mr) 5-10 cm	I, II
20* See Note XII	10-4	---	---	≥ 35 m	1.0	24-52	B (tg) 1-2 m + S (mr) 20-25 cm	I, V, VI
		---	---	< 35 m			B (tg) 1-2 m + S (mr) 10-20 cm	I, II. VI
21	4-1	≥ 12.5	≤ 0.75	---	1.5	2.1-6.5	B (utg) 1 m + S 2-3 cm	I
		< 12.5	≤ 0.75	---			S 2.5-5 cm	I
		---	> 0.75	---			B (utg) 1 m	I
22	4-1	> 10, < 30	> 1.0	---	1.5	4.5-11.5	B (utg) 1 m+clm	I
		≤ 10	> 1.0	---			S 2.5-7.5 cm	I
		< 30	≤ 1.0	---			B (utg) 1 m + S (mr) 2.5-5 cm	I
		≥ 30	---	---			B (utg) 1 m	I

TABLE A13 (Cont'd.)

Support Category	Q	RQD/J_n	J_r/J_n	SPAN/ESR	P Kg/cm^2 (approx.)	SPAN/ ESR (m)	Type of Support	Note: See Ref. A1
23	4-1	---	---	$\geq$ 15 m	1.5	8-24	B (tg) 1-1.5 m + S (mr) 10-15 cm	I, II, IV, VII
		---	---	< 15 m			B (utg) 1-1.5 m + S (mr) 5-10 m	I
24* See Note XII	4-1	---	---	$\geq$ 30 m	1.5	18-46	B (tg) 1-1.5 m + S (mr) 15-30 cm	I, V, VI
		---	---	< 30 m			B (tg) 1-1.5 m + S (mr) 10-15 cm	I, II, VII

*Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

(tg) = tensioned (expanding shell type for competent rock masses, grouted post-tensioned in very poor quality rock masses, see Note XI)

S = shotcrete

(mr) = mesh reinforced

clm = chain link mesh

CCA = cast concrete arch

(sr) = steel reinforced

Bolt spacings are given in metres (m). Shotcrete, or cast concrete arch thickness, is given in centimetres (cm).

TABLE A14

Support Measures for Rock Masses of "Very Poor" Quality (Q range: 1.0-0.1)

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_a	SPAN/ESR (m)	P kg/cm^2 (approx.)	SPAN/ESR (m)	Type of Support	Note: See Ref. A1
25	1.0-0.4	> 10	> 0.5	---	2.25	1.5-4.2	B (utg) 1m+mr or clm	I
		≦ 10	> 0.5	---			B (utg) 1m+S(mr) 5 cm	I
		---	≦ 0.5	---			B (tg) 1m+S(mr) 5 cm	I
26	1 0-0.4	---	---	---	2.25	3.2-7.5	B (tg) 1 m + S (mr) 5-7.5 cm	VIII, X, XI
		---	---	---			B (utg) 1m+S 2.5-5 cm	I, IX
27	1.0-0.4	---	---	≧ 12 m	2.25	6-18	B (tg) 1 m + S (mr) 7.5-10 cm	I, IX
		---	---	< 12 m			B (utg) 1 m + S (mr) 5-7.5 cm	I, IX
		---	---	> 12 m			CCA 20-40 cm + B (tg) 1 m	VIII, X, XI
		---	---	< 12 m			S (mr) 10-20 cm + B (tg) 1 m	VIII, X, XI
28*	1.0-0.4	---	---	≧ 30 m	2.25	15-38	B (tg) 1 m + S (mr) 30-40 cm	I, IV, V, IX
		---	---	≧ 20, < 30			B (tg) 1 m + S (mr) 20-30 cm	I, II, IV, IX
		---	---	< 20 m			B (tg) 1 m + S (mr) 15-20 cm	I, II, IX
		---	---	---			CCA (sr) 30-100 cm + B (tg) 1 m	IV, VIII, X, XI
29*	0.4-0.1	> 5	> 0.25	---	3.0	1.0-3.1	B (utg) 1 m + S 2-3 cm	---
		≦ 5	> 0.25	---			B (utg) 1 m + S (mr) 5 cm	---
		---	≦ 0.25	---			B (tg) 1 m + S (mr) 5 cm	---

TABLE A14 (Cont'd.)

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_n	SPAN/ESR(m)	P kg/cm^2 (approx.)	SPAN/ESR (m)	Type of Support	Note: See Ref. A1
30	0.4-0.1	= 5	---	---	3.0	2.2-6	B (tg) 1 m + S 2.5-5 cm	IX
		5	---	---			S (mr) 5-7.5 cm	IX
		---	---	---			B (tg) 1 m + S (mr) 5-7.5 cm	VIII, X, XI
31	0.4-0.1	4	---	---	3.0	4-14.5	B (tg) 1 m + S (mr) 5-12.5 cm	IX
		= 4,	= 1.5 --	---			S (mr) 7.5-25 cm	IX
		1.5	---	---			CCA 20-40 cm + B (tg) 1 m	IX, XI
		---	---	---			CCA (sr) 30-50 cm + B (tg) 1 m	VIII, X, XI
32	0.4-0.1	---	---	= 20 m	3.0	11-34	B (tg) 1 m + S (mr) 40-50 cm	II, IV, IX, XI
		---	---	20 m			B (tg) 1 m + S (mr) 20-40 cm	III, IV, IX, XI
		---	---	---			CCA (sr) 40-120 cm + B (tg) 1 m	IV, VIII, X, XI

*Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

A tunnel with given values of SPAN/ESR and quality Q will require reduced overall measures for temporary support. Appropriate reductions in support can be obtained by increasing the value of ESR to 1.5 ESR and by increasing from Q to 5 Q.

It should be emphasized that the above support recommendations are based for the most part on general engineering practice for a given type of excavation. If the quality of drilling and blasting is better or worse than normal, then the recommended support may be over-conservative or inadequate respectively.

Rock Structure Rating

The rock structure rating for the design of tunnel linings uses an empirical relationship between RSR values and support values by who developed a standard datum by which different supports could be compared (Ref. A9). The majority of case history tunnels were supported with steel ribs. A measure was used that would relate support installation to a theoretical support (rib size and spacing) which could be determined for each tunnel. This measure, the Rib Ratio (RR), was developed from Terzaghi's formula for roof loads in loose sand below the water table as datum conditions.

From the support for this datum condition for different tunnel diameters (Table A15), the RR for a particular tunnel study section is obtained by dividing the rib spacing by the actual rib spacing used in the study section and multiplying by 100.

Rib ratios were determined for each tunnel study section and used to develop the empirical relationship between determined RSR values and ground support requirements (Figure A4), resulting from the analysis of 190 tunnel sections; however, this relationship is not representative of

TABLE A15

Theoretical Spacing of Typical Rib Sizes for Datum Condition
(Ref. A9)

Rib Size	Tunnel Diameter 10'	12'	14'	16'	18'	20'	22'	24'	26'	28'	30'
417.7	1.16										
4H13.0	2.01	1.51	1.16	0.92							
6H15.5	3.19	2.37	1.81	1.42	1.14						
6H20		3.02	2.32	1.82	1.46	1.20					
6H25			2.86	2.25	1.81	1.48	1.23	1.04			
8WF31				3.24	2.61	2.14	1.78	1.51	1.29	1.11	
8WF40					3.37	2.76	2.30	1.95	1.67	1.44	1.25
8WF48						3.34	2.78	2.35	2.01	1.74	1.51
10WF49								2.59	2.22	1.91	1.67
12WF53										2.19	1.91
12WF65											2.35

Note: Spacing in feet.

Note: 12WF65; Standard wide flange steel section with a nominal depth of 12 in. and weight per lineal foot of 65 lb.

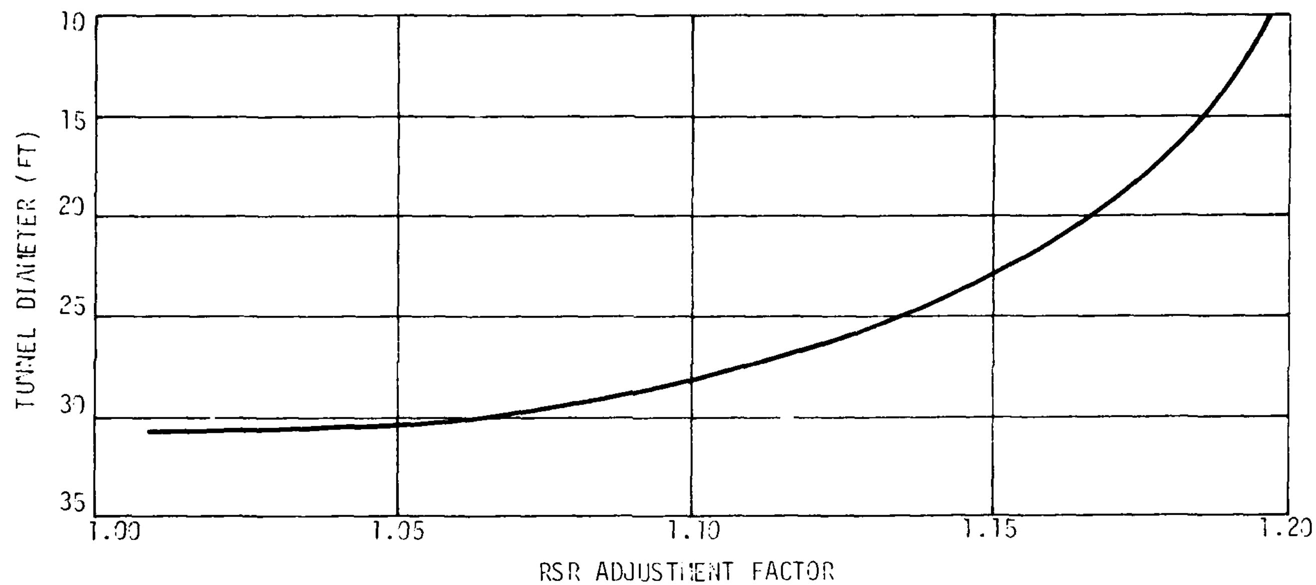

FIGURE A3 - RSR Adjustment for TBM Operation (Ref. A10)

special rock conditions such as swelling or squeezing rocks, the points for which are shown as circles (Figure A3).

Two important limitations of the correlation between RSR and RR are:

1. Case history data were used to develop the RSR and RR values. Since all the tunnel sections were supported safely, the factors of safety are unknown and it is probable that the sections were oversupported to some degree. Consequently, the correlation between RSR and RR could be conservative.

2. The relation between RSR and RR and the support requirements discussed below are for tunnels excavated by drill and blast. Because of the lack of data, the same relation could not be developed for machine driven tunnels; however, it is suggested that an upward adjustment to the RSR values (Figure A3) be made. This would adjust for the better condition of the rock structure resulting from the use of a TBM, and the normal correlation between RR and support requirements should be used with the adjusted RSR values.

Correlation Between RSR and Rock Load. The rib ratio RR is a measure of the rock loads as proposed by Terzaghi, and the relation between RSR and RR can be interpreted in terms of rock loads.

For a known RSR, the rock load, W_r, on the support system is given by (Table A16):

$$W_r = \left(\frac{D}{302}\right)\left(\frac{8800}{RSR + 30}\right) - 80$$

where

W_r = rock load (kips/sq ft)

D = diameter of tunnel (ft)

RSR = rock structure rating

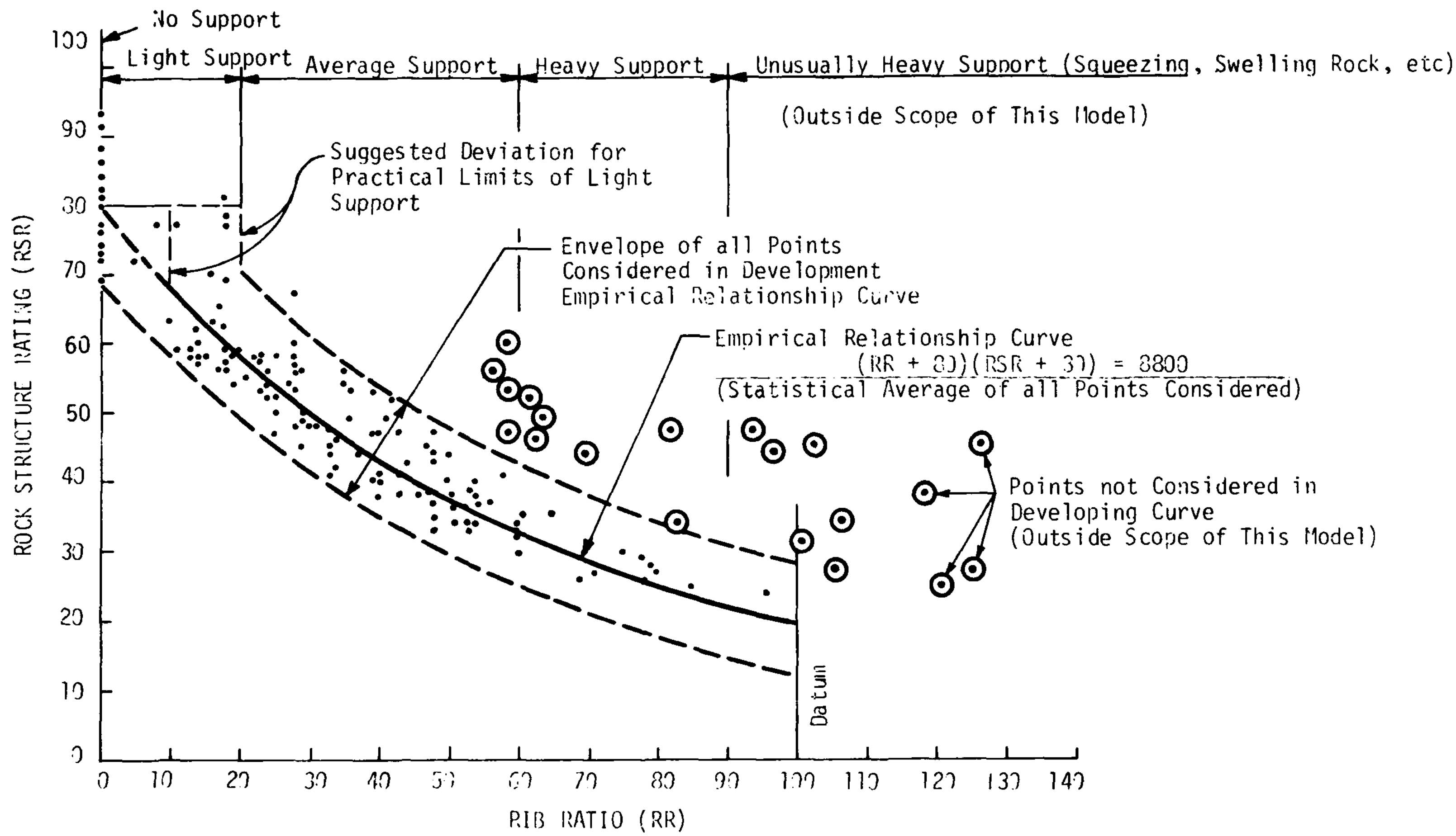

FIGURE A4 - Final Correlation of RSR and RR (Ref. A10)

TABLE A16

Correlation of Rock Structure Rating to Rock Load and Tunnel Diameter
(Ref. A9)

Tunnel Diameter (D)	Rock Load on Tunnel Arch (K/ft^2) 0.5	1.0	1.5	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
	Corresponding Values of Rock Structure Ratings (RSR)											
10'	62.5	49.9	40.2	32.7	21.6	13.8						
12'	65.0	53.7	44.7	37.5	26.6	18.7						
14'	66.9	56.6	48.3	41.4	30.8	22.9	16.8					
16'	68.3	59.0	51.2	44.7	34.4	26.6	20.4	15.5				
18'	69.5	61.0	53.7	47.6	37.6	29.9	23.8	18.8				
20'	70.4	62.5	55.7	49.9	40.2	32.7	26.6	21.6	17.4			
22'	71.3	63.9	57.5	51.9	42.7	35.3	29.3	24.3	20.1	16.4		
24'	72.0	65.0	59.0	53.7	44.7	37.5	31.5	26.6	22.3	18.7		
26'	72.6	66.1	60.3	55.3	46.7	39.6	33.8	28.8	24.6	20.9	17.7	
28'	73.0	66.9	61.5	56.6	48.3	41.4	35.7	30.8	26.6	22.9	19.7	16.8
30'	73.4	67.7	62.4	57.8	49.8	43.1	37.4	32.6	28.4	24.7	21.5	18.6

These rock loads can be used in a conventional way to design any type of support system including steel ribs, rock bolts, shotcrete, the Bernold System, or even a permanent concrete arch with a higher degree of reliability than that obtained by other empirical design methods.

Relation Between RSR and Ground Support Requirements. The relation between RSR and RR can be used as follows:

For a steel rib support from the tunnel diameter, the datum condition (i.e., the reference rib spacing for a given size of ribs) can be obtained (Table A15). From the value of RSR, the rib ratio RR can be determined from the average curve (Figure A3) or from the equation:

$$RR = \left(\frac{8800}{RSR + 30}\right) - 80$$

The necessary rib spacing is obtained by dividing the reference rib spacing by the computed RR and multiplying the result by 100. Typical results are given for support requirements for tunnels with diameters of 10 feet, 14 feet, and 20 feet (Figures A5, A6, & A7) together with design criteria for rock bolts and shotcrete. These have been established based on simplified assumptions from the rock loads (Table A16). For rock bolts, it is assumed that the spacing is related to the rock load and the allowable tensile force in the bolt by:

$$S = B/W_r$$

where

S = rock bolt spacing (ft)

W_r = rock load (kips/sq ft)

B = allowable tensile force in bolt (kips)

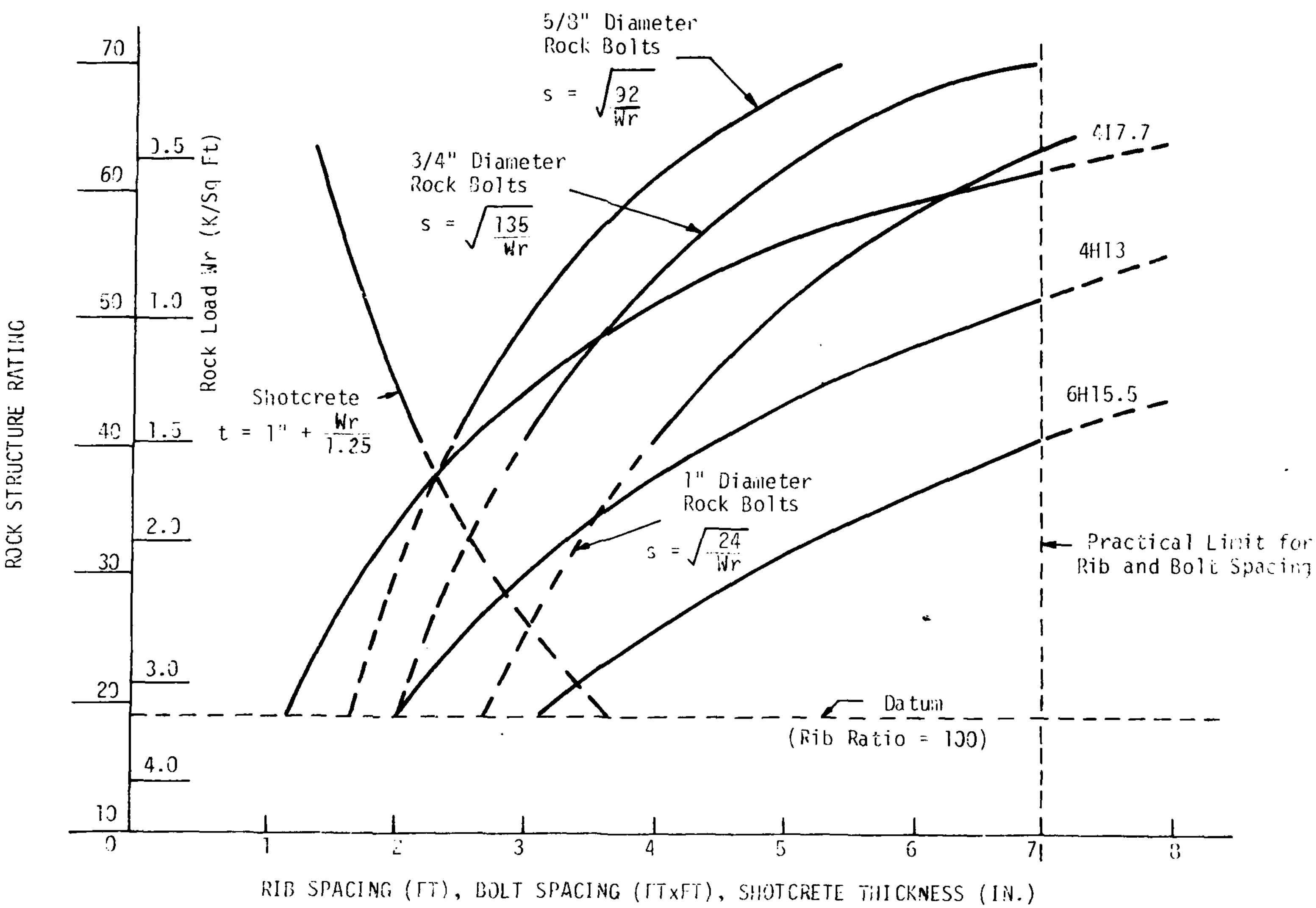

FIGURE A5 - Support Requirement Chart for a 10-Foot Diameter Tunnel (Ref. A10)

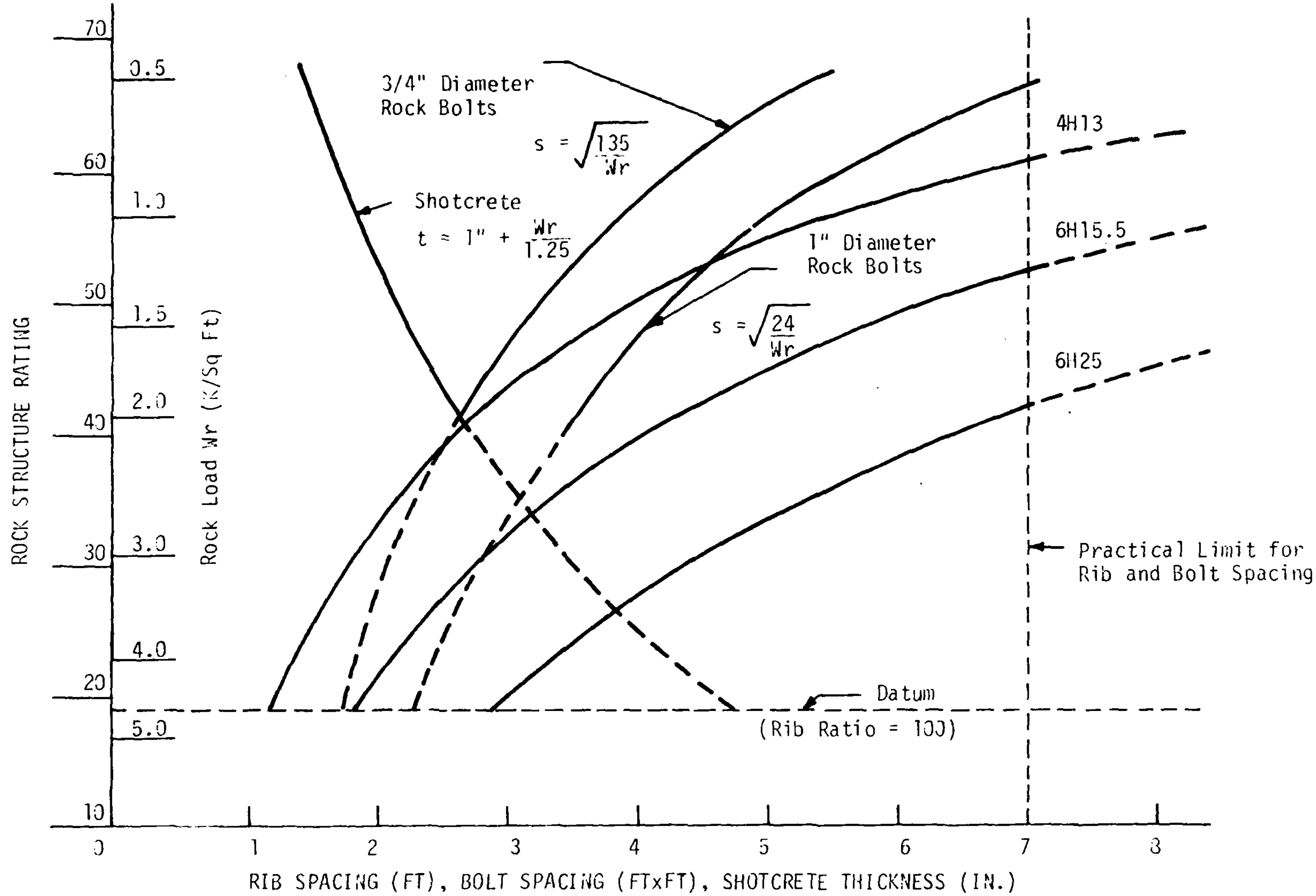

FIGURE A6 - Support Requirement Chart for a 14-Foot Diameter Tunnel (Ref. A10)

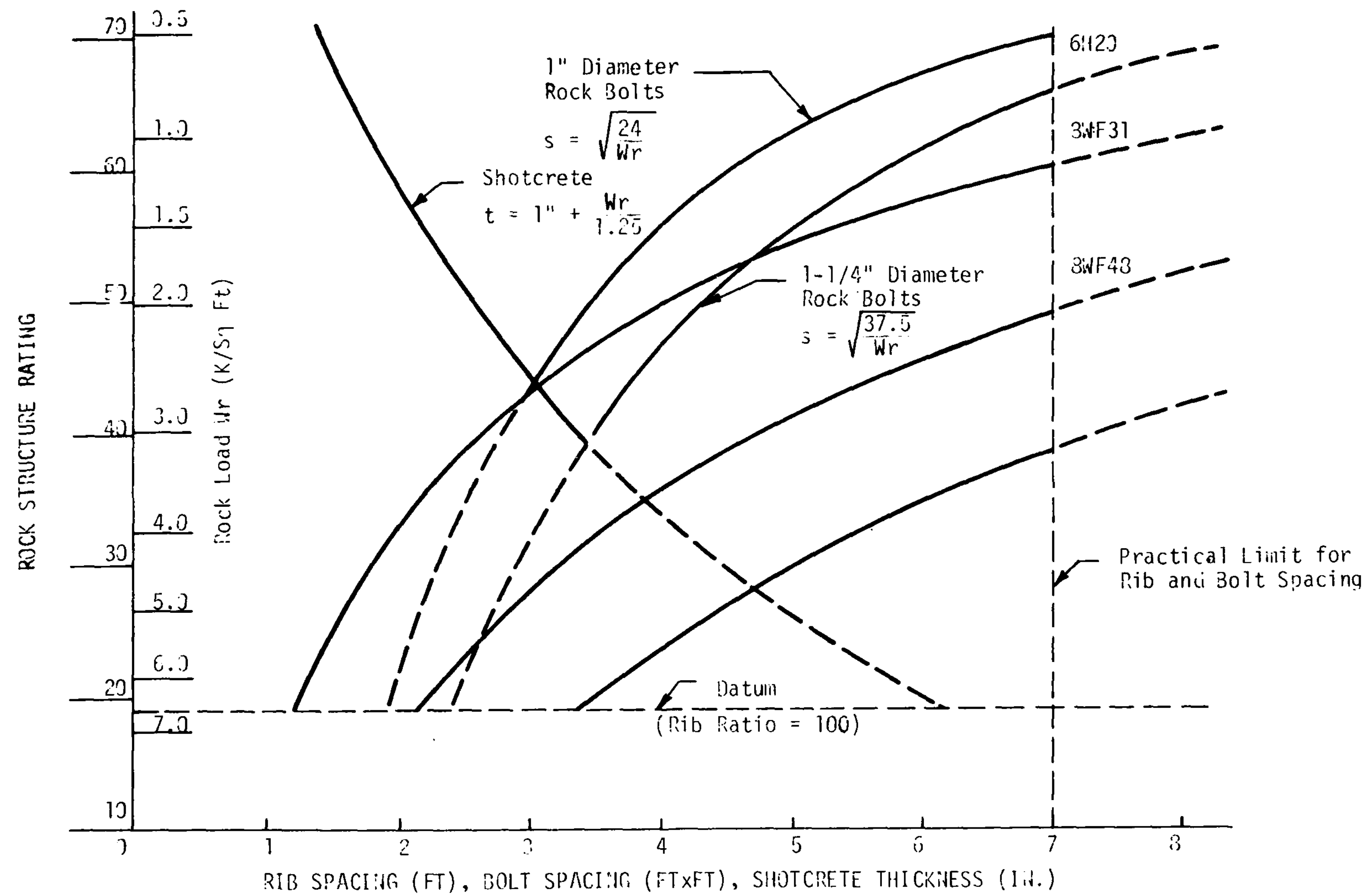

FIGURE A7 - Support Requirement Chart for a 20-Foot Diameter Tunnel (Ref. A10)

For shotcrete, the nominal thickness required is related to the rock load by:

$$t = 1 + (W_r/1 \cdot 25) \qquad \text{(A4)}$$

where

t = shotcrete thickness (in.)

W_r = rock load (kips/ft^2)

These are conservative assumptions in that the rock load is assumed to be the same as that for steel ribs; the effects of both rock bolts and shotcrete in reducing the rock loads are neglected. Thus, the design criteria may be over-conservative. Rock load reductions associated with the early placement of rock bolts or shotcrete should be considered. From the formula (Table A16 & Figures A5 to A7) supports can be designed for rocks with an RSR between 18 and about 75, with values less than 17 representing soft ground or soils, while values above 75 or 80 represent competent rocks which will require little or no support.

Summary of Rock Classification Methods. Rock classification of the methods are based on case histories, and if these tunnels were "over supported", the methods will perpetuate this over design. While the methods are based on rock conditions in drilled and blasted tunnels, a correction factor can be applied for machine driven tunnels, but the resulting designs may be of lesser quality. The RQD value is a useful parameter which is included in the CSIR and NGI methods but not in the RSR method. Some combination of these approaches, using the meaningful factors of each including the strength properties of the rock, would seem the best approach. The best current design base would be to use the RSR plus either or both the NGI and CSIR methods.

Currently, it is felt that the main advantage of the Rock Classification System is that it is based on rock properties which can be determined from a geotechnical investigation and allow the engineer to design the support system prior to the beginning of the tunnel excavation. Rock conditions vary over short distances and values of the quality index can be expected to differ locally from the average design values. As a result, changes in support should be expected during the progress of the tunnel excavation.

Secondary Linings for Tunnels in Rock

In many tunnels, the primary support installed immediately behind the face is sufficient to carry the rock loads for the full lifetime of the tunnel and a secondary lining may not be necessary. Secondary linings are required only if the tunnel is to be waterproofed, the surrounding rock is of a swelling type, or they are installed for aesthetic or psychological reasons as in transportation tunnels.

Where secondary linings do not have any structural function, their structural design is limited to the analysis of the stability under their own weight. Where the flow of water into a finished tunnel is too large to control by means of drains, it is necessary to provide a waterproofing membrane which will be subject to hydrostatic pressures and will require a competent support. A secondary lining can provide such support, the membrane being installed between the primary and secondary linings. The design of a secondary lining which is subjected to hydrostatic pressures needs sufficient strength to resist the resulting ring thrust plus the small moments due to its own weight. Also, a thickness sufficient to ensure its stability against buckling is required. Buckling can be calculated by:

$$P_{er} = (3\,E\,I/R^3) > P \qquad \text{(A5)}$$

where

P_{er} = critical pressure producing buckling of the lining (lb/ft^2)

E = modulus of elasticity of lining material (lb/ft^2)

I = moment of inertia of lining (ft)

R = radius of lining (ft)

p = applied hydrostatic pressure (lb/ft^2)

The design of the support system for a tunnel (in swelling rock) is related to a high degree of uncertainty since the cause, magnitude, and direction of the forces caused by the swelling process are unknown in most rock formations. If the swell potential of the rock is known, additional support may be provided by a secondary lining. Currently, the secondary lining is assumed to carry the full rock load and is designed accordingly. However, Deere, et al (Ref. A9), suggest that the secondary lining be assumed to carry only that part of the rock load which will develop after this lining is installed. The longer the time between the excavation of the tunnel and the installation of this lining, the smaller the load will be which must be carried by the secondary lining. Also, Deere, et al, suggest the use of frangible backpacking between the lining and the rock. In any case, the design of linings in swelling rock is based on local experience and will, therefore, be highly empirical.

Instrumentation and Observation

The above design approaches are largely empirical, and in some cases, possibly conservative. A rational and economic application of empirical

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A9. Deere, D.U., R.B. Peck, J.E. Monsees, and B. Schmidt, 1969, "Design of Tunnel Liners and Support Systems," Report for U.S. Department of Transportation, OHSGT Contract 3-0152, No. PB 183 799, NTIS, Springfield, Virginia, USA, p. 287.

Instrumentation and Observation

The above design approaches are largely empirical, and in some cases, possibly conservative. A rational and economic application of empirical design assumptions to local conditions, involving a program of instrumentation and observation of tunnel linings in sample design areas, is essential and should be initiated as soon as possible.

This program should be aimed at:

1. The evaluation of rock structure ratings based on local geological conditions, and correlated with rock quality indices such as RQD.

2. The evaluation of rock support requirements in terms of factors such as rock bolt loads and their variation with time, tunnel roof and wall movements as related to rock bolt spacing or shotcrete thickness, and

3. The measurement of deformation and ring loads in bolted segment linings in soft ground.

The implementation of such instrumentation program will permit rational applications of empirical design methods to local conditions and lead to modifications or improvements of linings. The end product desired is an economical lining consistent with safety, strength, and durability. It should be planned to incorporate in selected initial tunnel sections, trial support systems lighter than those calculated by existing RSR methods. These could be monitored as part of the program and results used to optimize support design.

Methods of Tunnel Lining

The behavior of lining in tunnels used for civilian purposes (static loading) is affected by (1) the characteristics of the discontinuities in the rock, (2) the time between excavation and placement of support, or

the "stand-up" time, (3) the method of excavation, either drill and blast which may weaken the rock immediately around the opening, or tunnel boring which causes a minimum of disturbance, and (4) the flexibility of the support system to allow enough rock movement so that arching may occur.

For the DBM, the lining will be designed to support not only the load due to the conventionally assumed types of static loading, but to offer some resistance to the loading caused by the stress wave generated by a nuclear detonation at the surface.

The types of lining or support include steel ribs, rock bolts, shotcrete, prefabricated concrete segments, and specialized types of liners. The type and strength of liners or support systems for the DBM system must have a minimum strength to support the static loads. The openings at or near the missile locations will probably be designed to give increased strength and resistance to collapse under static plus dynamic loading. Both primary and secondary linings may be used depending upon requirements.

Lining Design - Empirical Approaches

For static loading only, the load on the lining must be determined to give a basis for selection of the type and strength of the lining, which in turn will determine the method of installation and cost. One of the three methods of evaluating effects of geological factors discussed under Geotechnical Considerations may be used to determine the load factor.

Steel Ribs

Steel ribs, with timber lagging in blocky and seamy rock conditions, still find application in poor rock conditions, but their use is reduced considerably in favor of support systems, such as rock bolts and shotcrete.

Two types of steel support systems are generally used: continuous ribs, and full circle ribs. Continuous ribs are made of a pair of steel,

wide-flange (WF) beams, shaped to fit the tunnel opening and bolted together at the tunnel crown set on kicker foot blocks. Full circle ribs, for circular machine bored tunnels or in tunnels in squeezing, swelling, and crushed rock, are made of three or more circular-section beams. Placed at right angles to the centerline of the tunnel and at regular longitudinal spacing, the ribs, often referred to as sets, serve to distribute the rock load by ring action (full circle ribs).

The size of the beams and the rib spacing are determined by the local rock conditions and expected rock loads. The size of the beams ranges from 4-inch "I" beams, 7.7 pounds per foot (4I7.7) to 8-inch WF beams, 48 pounds per foot (8WF48) for tunnels with a diameter less than 20 feet. Rib spacings range from 1 to 7 feet, 4 feet being common.

When steel ribs are required, they are installed as soon as possible. For TBM excavated tunnels, this is done behind the back of the machine. To insure the development of the full strength of the steel ribs, the blocking points must be closely spaced and tightly wedged; uneven spacing of the blocks or a loosely wedged block could cause local bending or buckling or loosening of the rock due to lack of support. The support erection of steel is carried out by hand and the prospect of automatic erection methods being developed for this method is minimal.

The steel rib support system has two significant advantages:

1. Long experience has been gained from successful and unsuccessful applications, possible sources of malfunction have largely been identified, and proven empirical design criteria have been established.

2. The method is adaptable to any rock condition. The wide choice of rib sections and spacing allows selection of a support suitable for a wide

variety of rock loads. Steel ribs furnish all the necessary support without any direct contribution from the rock so that they can be installed even in very weak rock.

The disadvantages of steel rib supports are:

1. The installation of the wood blocking requires extreme care, and installation by hand is relatively slow. In machine excavated tunnels where the tunneling operation is continuous and the rate of machine advance is high, rib installation may be difficult to effect rapidly enough to avoid delays in excavation.

2. The steel rib system incorporates timber blocking and lagging which decay. This type of support is not permanent and must be complemented by a secondary permanent lining.

3. Steel ribs project 6 to 12 inches into the excavated tunnel sections. Therefore, the excavated section must be increased to allow for the space for the ribs and future secondary lining, which leads to increased costs.

Steel ribs are usually designed to resist rock loads computed by Terzaghi's "rock-load method" which applies only to wood-blocked steel sets, installed several feet behind the face of a tunnel excavated by conventional drilling and blasting techniques.

The "rock load," H_p, is the height of the mass of rock which tends to drop out of the roof of a tunnel. From an analysis of the amount of overbreak in various classes of rock and the geometric characteristics of the rock discontinuities and of the tunnel, Terzaghi (Ref. A2) estimated ranges of rock loads depending on the rock condition and the tunnel dimensions (Table A17). The rock load H_p, is given in terms of the width, B, and the height, H_t, of the tunnel. The lower limits of H_p depend upon favorable orientations of the rock discontinuities and the tunnel, the upper limits upon

TABLE A17

Rock Load H_p in Feet of Rock on Roof of Support in Tunnel with Width B (feet), and Height H_t (feet) at Depth of More Than 1.5 $(B + H_t)$

Rock Condition	Rock Load, H_p in ft	Remarks
1. Hard and intact	zero	Light lining, required only if spalling or popping occurs.
2. Hard stratified or schistose[2]	0 to 0.5 B	Light support.
3. Massive, moderately jointed	0 to 0.25 B	Load may change erratically from point to point.
4. Moderately blocky and seamy	0.25 B to 0.35 $(B + H_t)$	No side pressure.
5. Very blocky and seamy	(0.35 to 1.10) $(B + H_t)$	Little or no side pressure.
6. Completely crushed but chemically intact	1.10 $(B + H_t)$	Considerable side pressure. Softening effect of seepage towards bottom of tunnel requires either continuous support for lower ends of ribs or circular ribs.
7. Squeezing rock, moderate depth	(1.10 to 2.10) $(B + H_t)$	Heavy side pressure, invert struts required. Circular ribs are recommended.
8. Squeezing rock, great depth	(2.10 to 4.50) $(B + H_t)$	
9. Swelling rock	Up to 250 ft irrespective of value of $(B + H_t)$	Circular ribs required. In extreme cases use yielding support.

Notes: 1. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for types 4 to 6 can be reduced by fifty percent.

2. Some of the most common rock formations contain layers of shale. In an unweathered state, real shales are no worse than other stratified rocks. However, the term shale is often applied to firmly compacted clay sediments which have not yet acquired the properties of rock. Such so-called shale may behave in the tunnel like squeezing or even swelling rock.

If a rock formation consists of a sequence of horizontal layers of sandstone or limestone and of immature shale, the excavation of the tunnel is commonly associated with a gradual compression of the rock on both sides of the tunnel, involving a downward movement of the roof. Furthermore, the relatively low resistance against slippage at the boundaries between the so-called shale and rock is likely to reduce very considerably the capacity of the rock located above the roof to bridge. Hence, in such rock formations, the roof pressure may be as heavy as in a very blocky and seamy rock.

adverse conditions. The major drawback of this method is that it is based on a qualitative description of the rock condition.

It appears that the method generally results in workable and successful designs, althouth it does not give the loads in the supports. A few measurements have been made of the actual loads on steel ribs installed in rock. These suggest that Terzaghi's rock load method is conservative, particularly with modern methods of blasting and the use of tunneling machines.

Based on experience and improved understanding of rock behavior, Deere, et al (Ref. A10), proposed a more modern approach to the design of steel ribs. Their basic concept is that the support system installed in the tunnel be considered as a reinforcement to assist the rock in supporting itself rather than to support the rock. Their proposed criteria (Table A18) offer major improvements over Terzaghi's method, and are related to the use of RQD as an index of the rock condition, to the distinction between drilled and blasted and machine-excavated tunnels, and to the general reduction of rock loads. In establishing this method, Deere, et al, assumed that the support system is installed as close to the face as possible, i.e., about 2 to 4 feet, and that the steel ribs are properly erected and blocked. A comparison between Terzaghi's and Deere's rock loads and field measurements (Figure A8) shows that Deere's rock loads are about 20% smaller than Terzaghi's, but still are larger than 90% of observed loads. The saving on steel ribs obtained by using tunneling machines is clearly shown, where rock loads are reduced by 25%.

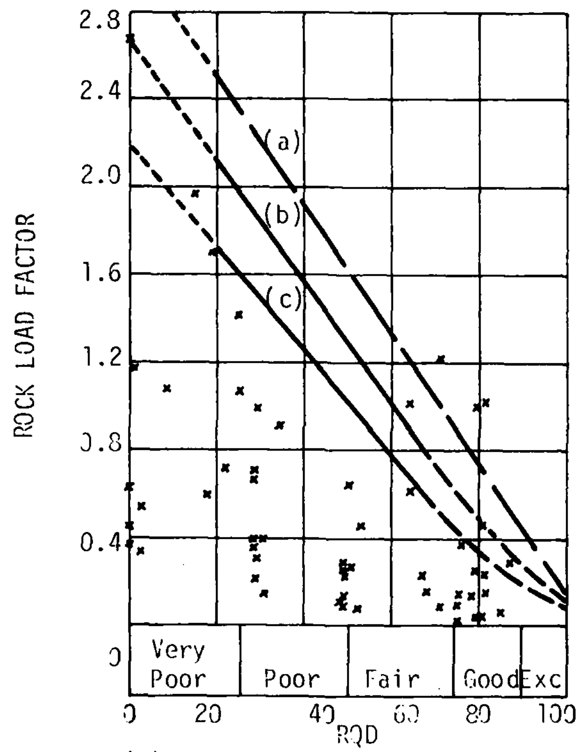
A10. Deere, D.U., J.E. Monsees, and B. Schmidt, 1970, "Design of Tunnel Liners and Support Systems," Highway Research Record No. 339, Transportation Research Board, Washington, D.C., USA.

TABLE A18

Guidelines for Selection of Steel Sets for 20 to 40-Foot Tunnels in Rock (Ref. A11)

Rock Quality	Construction Method	Steel Sets Rock Load (B = Tunnel Width)	Weight of Sets	Spacing
Excellent RQD > 90	Boring machine	(0.0 to 0.2)B	Light	None to occasional
	Drilling and blasting	(0.0 to 0.3)B	Light	None to occasional
Good RQD = 75 to 90	Boring machine	(0.0 to 0.4)B	Light	Occasional to 5 to 6 ft
	Drilling and blasting	(0.3 to 0.6)B	Light	5 to 6 ft
Fair RQD = 50 to 75	Boring machine	(0.4 to 1.0)B	Light to medium	5 to 6 ft
	Drilling and blasting	(0.6 to 1.3)B	Light to medium	4 to 5 ft
Poor RQD = 25 to 50	Boring machine	(1.0 to 1.6)B	Medium circular	3 to 4 ft
	Drilling and blasting	(1.3 to 2.0)B	Medium to heavy circular	2 to 4 ft
Very poor RQD < 25 (Excluding squeezing & swelling ground)	Boring machine	(1.6 to 2.2)B	Medium to heavy circular	2 ft
	Drilling and blasting	(2.0 to 2.8)B	Heavy circular	2 ft
Very poor, squeezing or swelling ground	Both methods	up to 250 ft	Very heavy circular	2 ft

Note: Table reflects 1969 technology in the United States. Groundwater conditions and the details of jointing and weathering should be considered in conjunction with these guidelines particularly in the poorer quality rock. See Deere, et al, 1969 for discussion of use and limitations of the guidelines for specific situations.



(a) Average for Terzaghi's Rock Load Factor

(b) Recommended for Steel Sets, Conventional Tunneling

(c) Recommended for Steel Sets, Machine Tunneling

x Field Measured Rock Loads

FIGURE A3 - Relationship of Rock Load Factors and RQD (Ref. A4)

Rock Bolts

The use of rock bolt support has developed over the last 20 years, particularly in machine bored tunnels. Considered at first as a replacement for steel ribs, but still as a temporary support, they are now being used for permanent support of tunnels.

Rock bolts provide tunnel support by reinforcing the rock mass to partially overcome its structural weaknesses. While rock bolts are used to provide a direct support to loose blocks or slabs, their application to small and intermediate sized tunnels is more often based on the "rock reinforcement" or "arch" concept.

Rock bolts usually consist of a steel rod, 0.5 to 1.5 inches in diameter, 6 to 10 feet long, installed in holes drilled into the tunnel roof and walls and tightened in place by means of an appropriate anchoring device at the end of the hole, and a plate and nut at the rock surface. To provide the necessary rock reinforcement, the rock bolts are installed on a regular pattern to form a continuous reinforced rock arch (Figure A9). In order for the arch to be formed, spacing of the bolts must be correctly designed and is a function of the rock quality and the size of the tunnel.

While rock bolts in current use consist of a rod of ductile steel, various types of anchoring systems are used. These are the split rod, expanding shells, resin bolts, and fiberglass reinforced polyester rock bolts.

The main advantage of rock bolts is related to the relative ease of installation, without undue interference with the excavation process. They also have an advantage in materials-handling, since the volume of materials handled is limited compared to a steel rib support system.

From the geotechnical standpoint, rock bolts are more satisfactory than steel ribs, since the inherent strength of the rock mass is used to help support the rock load. With bolt installation close to the tunnel

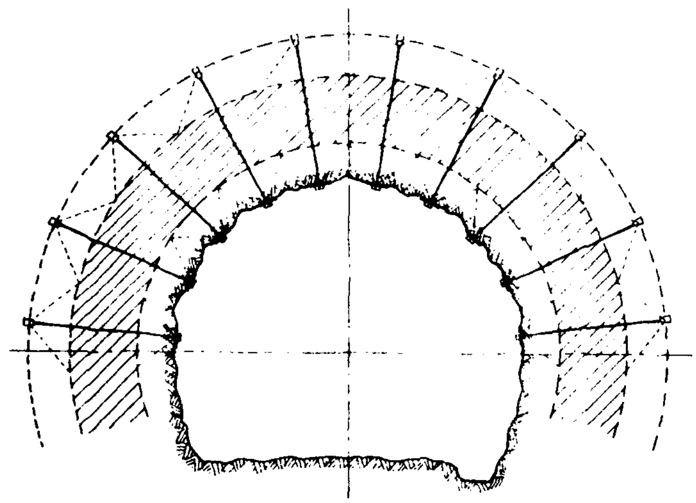


FIGURE A9 - Reinforced Rock Arch Formed by Rock Bolts (Talobre, J., La Mecanique Des Roches Dunod, Paris, 1957)

face, before significant loosening of the rock and with prestressing, almost the full rock strength can be utilized for the support of the tunnel opening. As a result, the necessary strength of the support system itself can be minimized. With minimum projection into the tunnel opening, rock bolts do not require enlargement of the tunnel diameter as is the case with steel ribs. Rock bolts can be made permanent by grouting to provide corrosion protection, and a secondary tunnel lining for structural reasons is often not necessary.

Despite the apparent advantages of rock bolts, the support is still discontinuous and isolated falls of rock blocks may occur. This risk can be eliminated by providing a continuous coverage of the roof by wire mesh or concrete, thus, in effect, installing a secondary lining.

Further, rock bolts cannot be easily used in badly broken rock where anchorage is difficult to obtain and where the bolt spacing would have to be so small as to be uneconomical. In such cases, steel ribs are usually preferred.

Finally, in spite of important progress to date, the design of a rock bolt system is largely empirical and coupled with a higher degree of uncertainty than that for steel ribs. However, the development of useful rock classification systems, correlated with experience in tunneling in different rock conditions, can be expected to reduce the uncertainty.

The understanding of the behavior of rock bolts and bolted tunnels is still limited so a rock bolt system is not designed but is selected based on rules of thumb and on tentative empirical considerations. That is, the size, type, and pattern of bolting is selected based on the best geologic information available, previous experience under similar conditions, and engineering judgement.

Typical design features are as follows:

1. The length of rock bolts should be greater than 1/3 of the tunnel width.

2. The spacing between rock bolts should always be less than the length.

3. Bolts should be installed in a regular pattern. Spacings between 3 and 8 feet are common practice.

4. Bolts should be installed as close to the face and as soon as possible, and should be pretensioned to ensure maximum efficiency.

Deere, et al (Ref. A11) have proposed typical bolting schemes applicable to the different possible rock properties (Table A19) which are applicable to bolts installed within 4 feet of the face and pretensioned. Goodman and Ewoldsen (Ref. A12) state that pretensioning of rock bolts minimizes rock loads and deformation provided the pretension corresponds to an applied rock bolt wall pressure of at least 10% of the initial rock stress (the rock bolt wall pressure is defined as the pretensioning force in the bolts divided by the area of the bolted wall). Rock bolts are usually superior to steel sets from both a technical and an economic point of view.

Shotcrete

Tunnel support by means of shotcrete is accomplished by applying a layer (4 to 8 inches) of concrete against the roof and walls of the tunnel in one pass by means of a pneumatic gun. The concrete mix is designed to develop an initial set within a few hours and a high strength within about 24 hours. The following factors are characteristic (Ref. A10):

A11. Goodman, R.E. and H.M. Ewoldsen, 1969, "A Design Approach for Rock Bolt Reinforcement in Underground Galleries," International Symposium on Large Permanent Underground Openings, Oslo, Norway.

TABLE A19

Guidelines for Selection of Rock Bolts for 20- to 40-Foot Tunnels in Rock (Ref. All)

Rock Quality	Construction Method	Rock Bolts (Conditional use in poor & very poor rock) Spacing of Pattern Bolts	Additional Requirements & Anchorage Limitations
Excellent RQD > 90	Boring machine	None to occasional	Rare
	Drilling and blasting	None to occasional	Rare
Good RQD = 75 to 90	Boring machine	Occassional to 5 to 6 ft	Occasional mesh and straps
	Drilling and blasting	5 to 6 ft	Occasional mesh or straps
Fair RQD = 50 to 75	Boring machine	4 to 6 ft	Mesh and straps as required
	Drilling and blasting	3 to 5 ft	Mesh and straps as required
Poor RQD = 25 to 50	Boring machine	3 to 5 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.
	Drilling and blasting	2 to 4 ft	Anchorage may be hard to obtain. Considerable mesh and straps required.
Very poor RQD < 25 (excluding squeezing and swelling ground)	Boring machine	2 to 4 ft	Anchorage may be impossible. 100 percent mesh and straps required.
	Drilling and blasting	3 ft	Anchorage may be impossible. 100 percent mesh and straps required.
Very poor, squeezing or swelling ground	Both methods	2 to 3 ft	Anchorage may be impossible. 100 percent mesh and straps required.

Note: Table reflects 1969 technology in the United States. Groundwater conditions and the details of jointing and weathering should be considered in conjunction with these guidelines particularly in the poorer quality rock. See Deere, et al, 1969 for discussion of use and limitations of the guidelines for specific situations.

1. Shotcrete is forced into open joints, fissures, seams, and irregularities in the rock surface and in this way serves the same binding function as mortar in a stone wall.

2. Shotcrete hinders water seepage from joints and seams in the rock and thereby minimizes piping of joint filling materials. Also, by sealing the rock surface, it prevents or reduces deterioration of the rock by air and water.

3. The adhesion of shotcrete to the rock surface together with the shear strength of the shotcrete layer provides considerable resistance to the fall of loose rock blocks from the roof of the tunnel.

4. Even at thicknesses of 4 to 8 inches, shotcrete provides structural support often of sufficient strength to support the rock mass.

Since shotcrete requires a certain time to develop its full strength, its use is limited to rock structures having a stand-up time longer than the setting time of shotcrete. It can be used either along or in combination with other support systems, such as rock bolts. Wire mesh reinforcing can also be used, but it often causes problems during installation. Tne wire mesh tends to vibrate during shotcrete application and frequently causes loosening of the concrete mass and lower strength.

The design of shotcrete support consists of selection of the appropriate concrete mix, the installation method, and the thickness. For design criteria in North America, see References 10, 13, and 14.

A12. Kobler, H.G., 1966, "Dry Mix Coarse Aggregate Shotcrete as Underground Support," ACI Special Publication No. 14, American Concrete Institute, Detroit, Michigan, USA.

A13. Mason, E.E., May 1968, "The Function of Shotcrete in Support of Lining of the Vancouver Railway Tunnel," paper presented at the Tunnel and Shaft Conference, University of Minnesota, Minnesota, USA.

Empirical design methods developed in Europe are based on possible failure mechanisms. The most common failure occurs where the weight of a loose block of rock in the tunnel roof exceeds the shear strength of the shotcrete layer along the limits of the rock block. Based upon experience, rough rules of thumb have been developed in various European countries: in Austria the shotcrete thickness is generally taken as 1/40 to 1/50 of the tunnel diameter; in Sweden 3 cm to 8 cm (1-1/4 to 3-1/4 inches) are applied immediately behind the face in jointed rock; in Germany 10 cm (4 inches) is considered adequate for tunnels up to 10 m in diameter. Such rules cannot be applied in rock types where no experience is available, but the shotcrete thicknesses recommended by Linder (Ref. A15), which are related to Lauffer's rock classification (Ref. A3), may be used (Figure A10). Consideration should be given to the fact that Reference A3 is based on rock conditions in Austria and that these conditions are identified from a qualitative point of view only.

In spite of limited experience, Deere, et al (Ref. A10), proposed empirical design criteria for shotcrete. Support requirements are based on the RQD (Table A20). The recommended shotcrete thicknesses are probably conservative.

The fundamentals of shotcrete mix design by Kobler (Ref. A13) indicate that in addition to strength requirements, the setting time is an important factor because it must be related to the stand-up time of the rock mass. The mix used in the Vancouver railroad tunnel (Ref. A14) may be considered as a reference; it consisted of 650 lb of Type 1 Portaland cement; 1,520 lb of sand; 850 lb of 1/4-in. stone, and 900 lb of 3/4-in. stone per cubic yard of concrete with a water/cement ratio of 0.35. In

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A14. Linder, R., 1963, "Spritzbeton im Felshohraumbau," Die Bautechnik, Vol. 40, No. 10, Berlin, Germany.

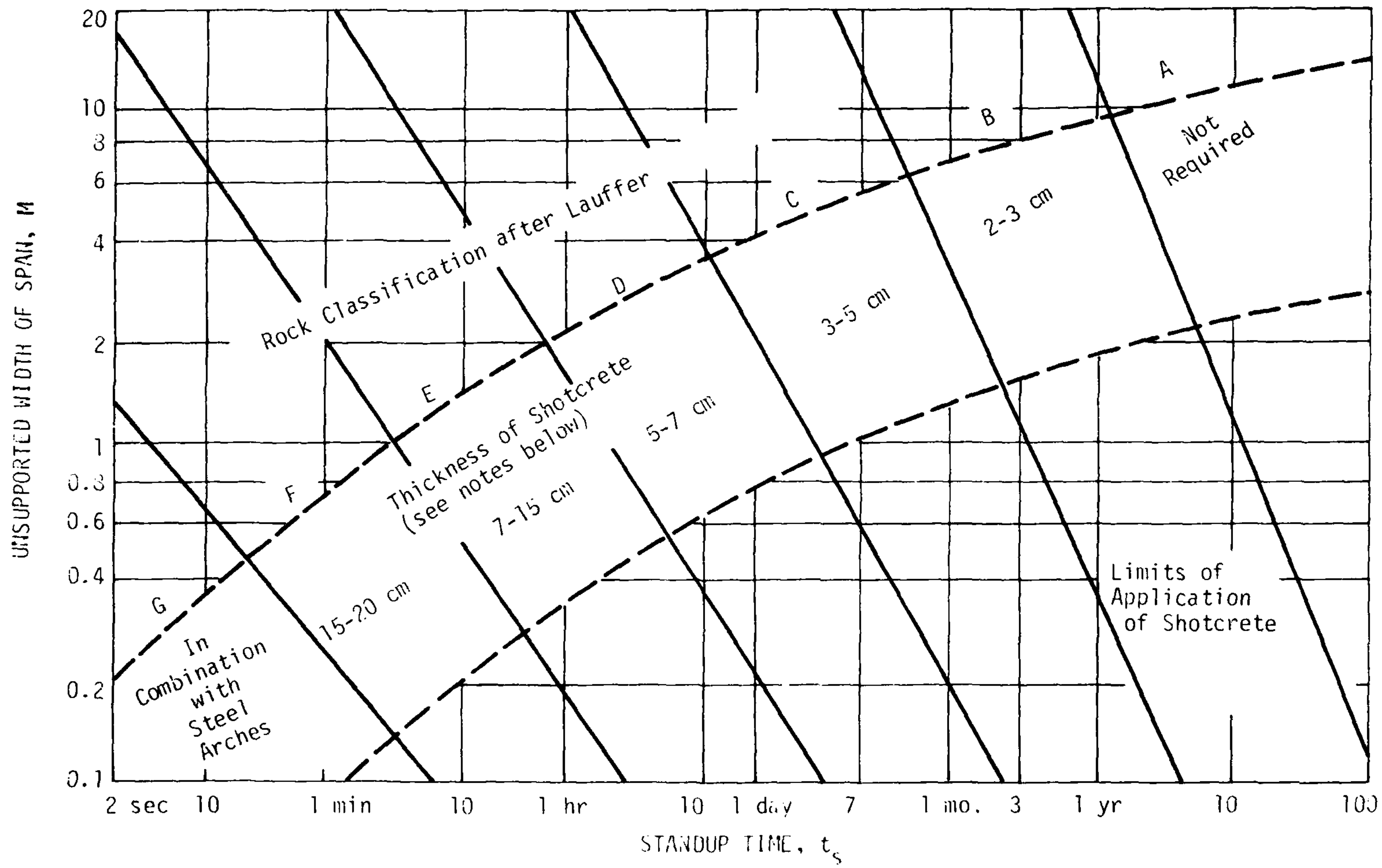


FIGURE A10 - Rock Reinforcement with Shotcrete from Linder (Ref. A4) (See Notes on Following Page)

Notes from Figure A10

(B) Alternatively rock bolts on 1.5 - 2 m spacing with wire net, occasionally reinforcement needed only in arch.

(C) Alternatively rock bolts on 1 - 1.5 m spacing with wire net, occasionally reinforcement needed only in arch.

(D) Shotcrete with wire net; alternatively rock bolts on 0.7 - 1 m spacing with wire net and 3 cm shotcrete.

(E) Shotcrete with wire net; rock bolts on 0.5 - 1.2 m spacing with 3 - 5 cm shotcrete sometimes suitable; alternatively, steel arches with lagging.

(F) Shotcrete with wire net and steel arches; alternatively strutted steel arches with lagging and subsequent shotcrete.

(G) Shotcrete and strutted steel arches with lagging.

TABLE A20

Guidelines for Selection of Shotcrete for 20- to 40-Foot Tunnels in Rock (Ref. All)

Rock Quality	Construction Method	Shotcrete (Conditional use in poor & very poor rock) Total Thickness Crown	Sides	Additional Support
Excellent RQD > 90	Boring machine	None to occasional local application	None	None
	Drilling and blasting	None to occasional local application 2 to 3 in.	None	None
Good RQD = 75 to 90	Boring machine	Local application 2 to 3 in.	None	None
	Drilling and blasting	Local application 2 to 3 in.	None	None
Fair RQD = 50 to 75	Boring machine	2 to 4 in.	None	Provide for rock-bolts
	Drilling and blasting	4 in. or more	4 in. or more	Provide for rock-bolts
Poor RQD = 25 to 50	Boring machine	4 to 6 in.	4 to 6	Rockbolts as required (4-6 ft cm^3)
	Drilling and blasting	6 in. or more	6 in. or more	Rockbolts as required (4-6 ft cm^3)
Very poor RQD < 25 (Excluding squeezing & swelling ground)	Boring machine	6 in. or more on whole section		Medium sets as required
	Drilling and blasting	6 in. or more on whole section		Medium to heavy sets as required
Very poor, squeezing or swelling ground	Both methods	6 in. or more on whole section		Heavy sets as required

Note: Table reflects 1969 technology in the United States. Groundwater conditions and the details of jointing and weathering should be considered in conjunction with these guidelines, particularly in the poorer quality rock. See Deere, et al, 1969 for discussion of use and limitations of the guidelines for specific situations.

addition, approximately 25 lb of accelerator, either "Tricosal TIKA" or "Sika Sigunit," were added. This mix produced a shot with the following strengths:

2 hours	200 lb/in^2
12 hours	800 lb/in^2
24 hours	1500 lb/in^2
28 days	4000 lb/in^2

The quality of shotcrete depends on the mix proportions as with conventional concrete, but shotcrete has a very low water/cement ratio, ranging from about 0.32 to 0.40.

To achieve a satisfactory bond and the rapid setting time required for tunnel support, various patented accelerating admixtures have been developed. These cause the concrete to achieve high strengths in a matter of hours. Compared to conventionally poured concrete, shotcrete is strong in compression and unusually so in flexure. (The "Vancouver mix" produced shotcrete with an average 28-day compressive strength of 4,000 to 5,000 lb/in^2 with an average 28-day modulus of rupture in flexure of 1,150 lb/in^2.) The high strength is probably caused by the low water/cement ratios and also by the high degree of compaction.

Details concerning various accelerators are given in Parker, et al (Ref. A16), Singh and Bortz (Ref. A17), and Anderson and Poad (Ref. A18).

A16. Parker, H.W., R.M. Semple, A. Rokhsar, E. Febres-Codero, D.U. Deere, and R.B. Peck, 1971, "Innovations in Tunnel Support Systems," Report prepared by the Dept. of Civil Engineering, University of Illinois, for the Office of High Speed Ground Transportation, Washington, D.C., USA.

A17. Singh, M.M. and S.A. Bortz, 1974, "Use of Special Cements in Concrete," Symposium on Use of Shotcrete for Underground Structural Support, Engineering Foundation in conjunction with ASCE and ACI, New York, N.Y., USA.

A18. Anderson, G.L. and M.E. Poad, 1974, "Early Age Strength Properties of Shotcrete," Symposium on Use of Shotcrete for Underground Structural Support, ACI Publication SP-45, American Concrete Institute, Detroit, Michigan, USA.

The selection of the applicable proportions for the mix and the accelerator depends upon the rate of advance and rock conditions.

Common rates for application are about 5 to 7 cubic yards per hour. For a tunnel about 15 feet in diameter, possibly requiring shotcrete about 6 inches thick, the application rate for this thickness of shotcrete would be approximately equal to the advance rate for excavation. Under difficult ground conditions, the shotcreting must be fully integrated with the excavation and material-handling sequence. High rates of application have been developed and used with larger equipment. Hence, it may be economical to delay shotcreting (providing that the stand-up time is adequate) to the end of the excavation shifts and to shotcrete during a maintenance shift.

An advantage of shotcrete is that it provides a continuous strong, yet yielding support. If it can be installed immediately behind the tunnel face, it leads to a significant reduction of rock loosening, rock pressure, and support requirements. It prevents air-slaking and moisture entrance into the rock and is thus effective for rocks sensitive to air and moisture as well as for swelling rocks.

Shotcrete is adaptable to changing rock conditions. One application is for rock where rock bolts alone are not adequate to support the rock, i.e., in rock of medium to poor quality. In general, shotcrete is an excellent final tunnel lining as well as rock support, and no secondary lining is required. Corresponding savings can be achieved not only in the cost of the support and lining system, but also savings due to the smaller size of the excavated tunnel.

One principal disadvantage of shotcrete was the lack of experience in North America in its use. Further construction data is required to confirm design methods and costs.

A new lining system has been developed recently (1976) in Switzerland, the "Bernold System," which consists of a concrete lining reinforced by special steel sheets that can be erected close to the tunnel face. It serves both as an intermediate tunnel support and as a permanent lining.

The main element in this is a steel sheet which is corrugated and "opened" (Figure A11). The sheets have a standard size of 3 feet 6 inches by 4 feet and are available in three thicknesses, 10, 14, and 17 gauge. Their shape depends on the radius of the tunnel and the design thickness of concrete lining. To form a continuous lining, the sheets are erected with a 4-inch overlap and special connections.

The sheets are usually erected in a vault close behind the tunnel face with temporary support being provided by steel sets. The space between the vault and the rock is filled with pumped concrete. After the concrete is set, the steel sets are removed and if required, the inner surface of the sheets is covered with concrete or gunite. If the rock surface is smooth, as with machine excavated tunnels, the sheets can be erected in contact with the rock and gunite can be used to ensure contact with the rock and continuous coverage of the sheets. The sheets, therefore, serve as a temporary protection against rock falls, as a form for the pumped concrete, and as reinforcement for the finished concrete arch.

The system has all of the advantages of shotcrete, and in addition, provides an increased shear strength to prevent roof blocks from falling. They also ensure an immediate support in the same manner as steel ribs in rocks with short stand-up times. The system is, therefore, suited for rocks of lower quality where it represents a worthwhile alternative to steel ribs.

Bernold Systems are designed generally on the basis of Terzaghi's rock loads, the system being considered as a thin arch. Full scale tests in

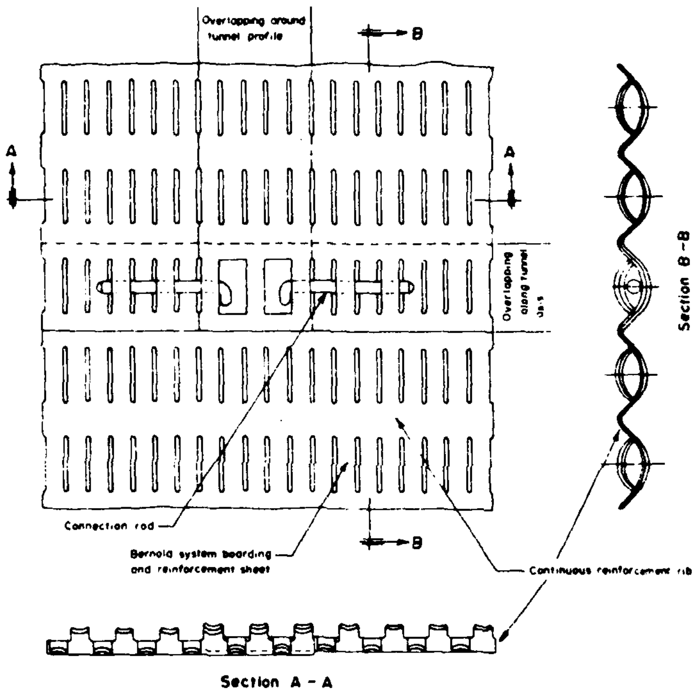


FIGURE A11- Bernold System Boarding and Reinforcement Sheet (Bernold, Jean, 1970; given in Nussbaum).

Switzerland and Japan have shown that a Bernold lining had a load-carrying capacity 30 to 50 percent greater than that of an unreinforced concrete lining of the same thickness. This system should be investigated further for possible use in the proposed DBM tunnel system.

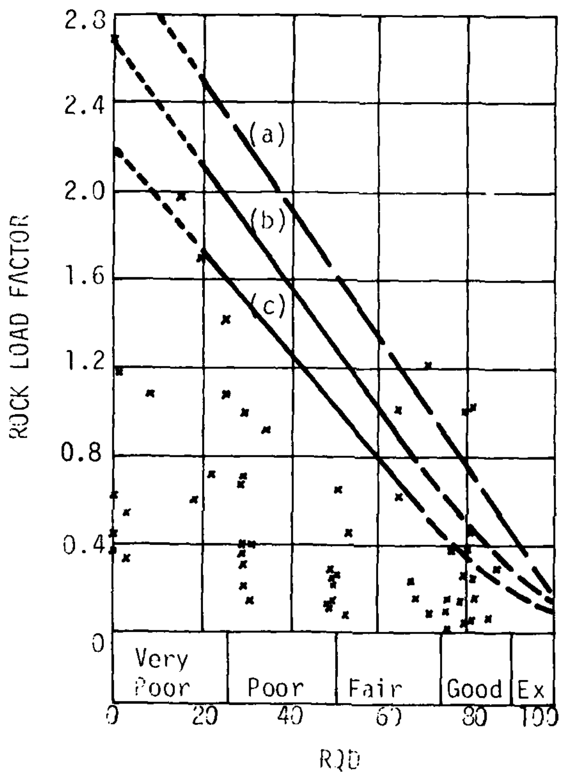
Lining Design and Rock Classification Systems

The rock classification systems described earlier were developed to assist in the design of lining systems for tunnels of varying sizes in different types of rock. The methods of their application are described below, and will then be employed to evaluate possible rock conditions and the support requirements for the DBM system.

These methods of design are just as empirical and are based upon experience. Because of the varied nature of rocks (anisotropic, nonhomogeneous, neither elastic nor plastic because of joints), the approach of rock classification based on observed and measured properties is the most promising basis for the design of linings for tunnels.

An attempt has been made by the originators of all of the systems described (RQD, CSIR, NGI, RSR) to relate the "Rating Figure" to the "Rock Load" as defined originally by Terzaghi (1946). Once the loads which the lining must carry are known or estimated, the design of the linings becomes a soluble structural problem.

RQD Method. Reference A10 has related the rock quality designation (RQD) to Terzaghi's Rock Load Factor (Figure A12), and also gives the author's recommended reductions in this factor based on rock loads measured in tunnels for conventional tunneling methods and machine tunneling, which causes less disturbance to the rock than drilling and blasting. Reference A12 gives three tables relating RQD to the design of steel sets (ribs), rock bolts, and shotcrete for tunnels in rock 20 to 40 feet in



(a) Average for Terzaghi's Rock Load Factor

(b) Recommended for Steel Sets, Conventional Tunneling

(c) Recommended for Steel Sets, Machine Tunneling

x Field Measured Rock Loads

FIGURE A12 - Relationship of Rock Load Factors and RQD (Ref. A4)

diameter (see Tables A18, A19, & A20). A further design aid for shotcrete given by Deere, et al (Ref. A11) (Figure A13) is based on Lauffer's classification of rock.

CSIR Method. Bieniawski has related his rating of rock quality to the selection of primary support for tunnels from 5 to 12 meters in diameter constructed by drilling and blasting in Table A21. This gives suggested support measures for rock bolts, shotcrete, and steel sets. These recommendations are based on experience with South African rocks.

NGI Method. This classification system is not yet a complete design system. However, it is a useful guide. Barton, et al (Ref. A3), relate the rock mass quality, Q, to the required support pressure for the roof as shown in Figure A13, which is explained as follows.

An empirical equation relating permanent support pressure and rock mass quality Q, which fits available case records quite well, was found to be

$$P_{roof} = (2/J_r)\ Q^{-1/3} \tag{A6}$$

where

P_{roof} = permanent roof support pressure in kg/cm^2

J_r = joint roughness number

Q = rock mass quality

The diagonal lines in Figure A13 are numbered with their respective J_r values using the above equation. The shaded envelope is an estimate of the range to be expected in practice from available case records. This utilizes a double dependence of support pressure on joint roughness number J_r, which appears to be realistic according to available case records.

While equation (A6) fits the data for case studies quite well, it was improved for a situation where the number of joint sets (J_n) falls below three, whence,

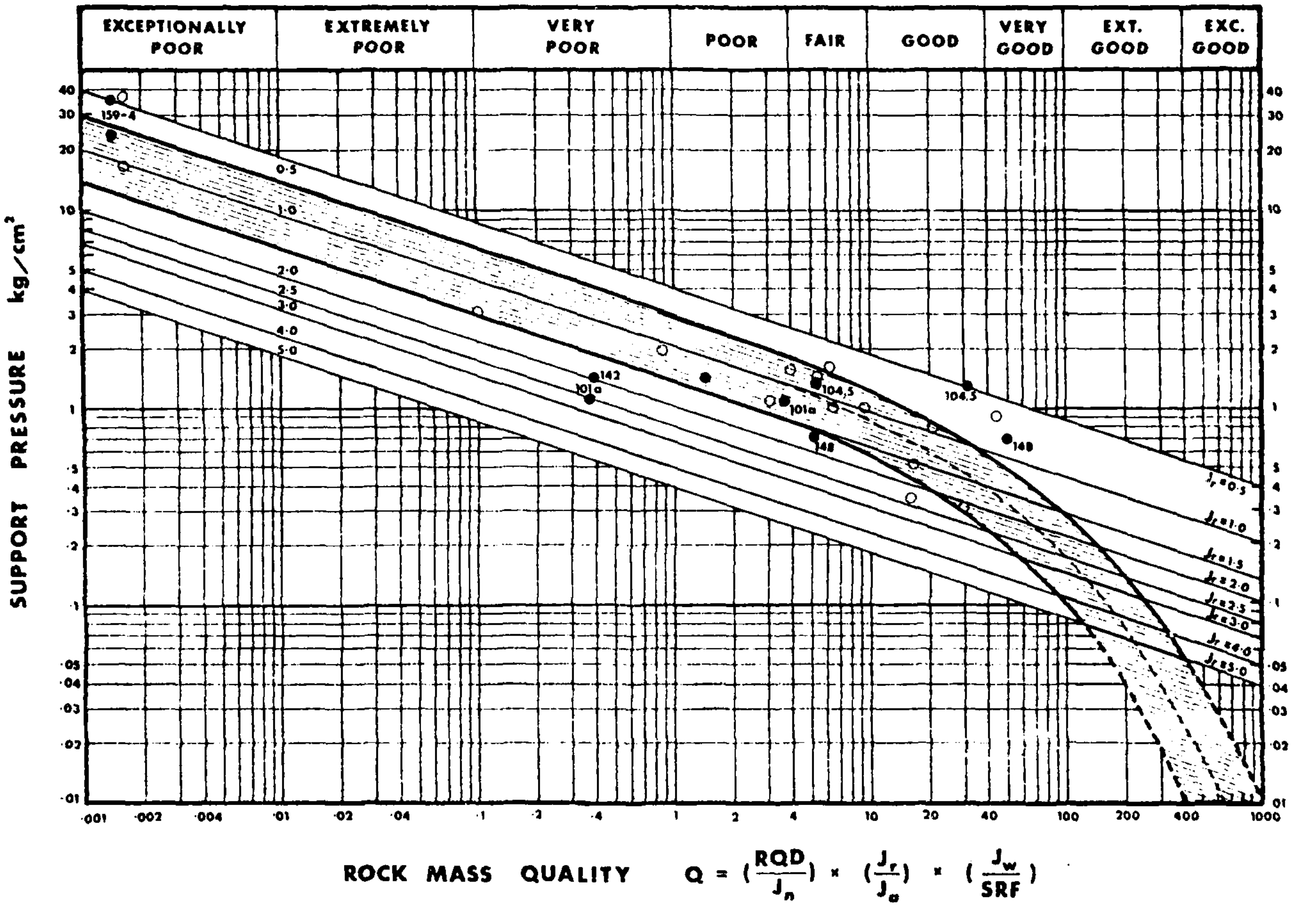


FIGURE A13 - Empirical Method of Estimating the Support Pressure (Barton, et al, 1974).

TABLE A21

Support Systems for Different Rock Mass Classes
(Ref. A4)

Rock Mass Class	Average Stand-Up Time at Unsupported Span	Alternative Support Systems: Rockbolts*		Shotcrete			Steel Sets	
		Spacing	Additional Support	Crown	Sides	Additional Support	Type	Spacing
1	10 years 5 m	Generally not required						
2	6 months 4 m	1.5-2.0 m	Occasional wire mesh in crown	50 mm	Nil	Nil	Uneconomic	
3	1 week 3 m	1.0-1.5 m	Wire mesh, plus 30 mm shotcrete in crown as required	100 mm	50 mm	Occasional wire mesh & rockbolts if necessary	Light sets	1.5-2.0 m
4	5 hours 1.5 m	0.5-1.0 m	Wire mesh, plus 30-50 mm shotcrete in crown & sides	150	100	Wire mesh & 3 m rockbolts at 1.5 m spacing	Medium sets plus 50 mm shotcrete	0.7-1.5 m
5	10 min. 0.5 m	Not recommended		200 mm	150 mm	Wire mesh, rockbolts & light steel sets Seal face. Close invert	Heavy sets with lagging, immediately 80 mm shotcrete	0.7 m

*Bolt diameter 25 mm, length 1/2 tunnel width. Resin bonded fully.

$$P_{roof} = \frac{2\, J_n^{1/2}\, (Q)^{-1/3}}{3 J_r} \qquad (A7)$$

From a large number of case histories, the rock quality Q was plotted against "equivalent dimensions" and the lower line in Figures A14 and A15 was found to be the approximate boundary between self-supporting excavations and those requiring some form of support, the equation for this line being given by

$$D_e' = 2\, Q^{0.4} \qquad (A8)$$

where

D_e' = the limiting value of SPAN/ESR

Q = the rock mass quality

Such spans range from 1.2 to 100 meters (Table A22). The span width is not necessarily the limiting factor, but the rock strength, lack of joints, a favorable stress field, and other factors are also important.

In order to identify important variations in support requirements, the factors RQD/J_n and J_r/J_a as well as Q (Ref. A8), the first of two excavations with the same values of Q may be bolted, and in the second, only shotcreted. The factor RQD/J_n which describes the block size will normally separate these two cases. That is, rock masses with RQD/J_n values larger than 10 will likely be massive to blocky requiring only bolting, while values less than 10 may represent blocky jointed rock, which can often require only shotcrete. In other cases, the factor J_r/J_a (which describes interblock shear strength) may be more important. Also, the equivalent dimension (D_e), which is equal to SPAN/ESR, is a third factor accounting for differences in support practice.

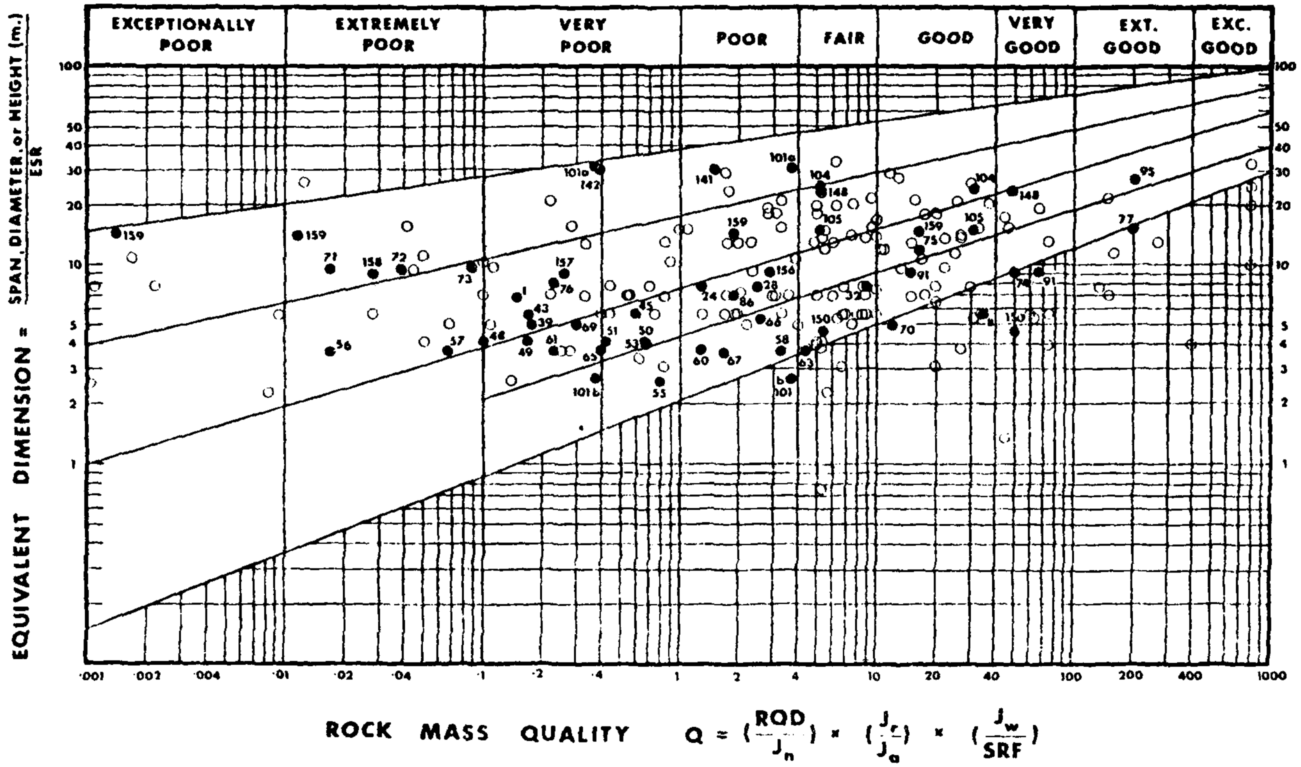
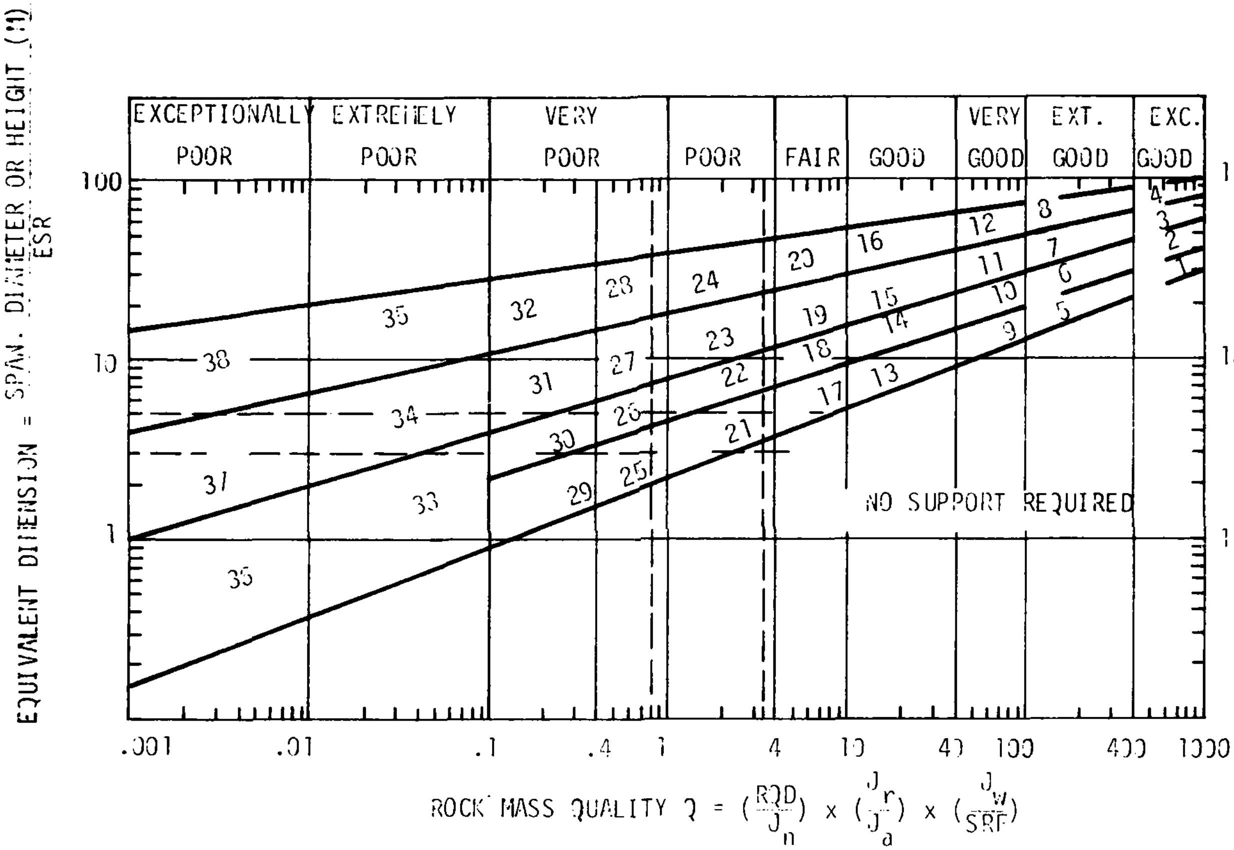


FIGURE A14 - Support Recommendations Based on the Analyses of More Than 200 Case Records (Barton, et al, 1974).



NOTE: Dashed Lines Show Ontario Conditions

FIGURE A15 - Tunnel Support Chart Showing the Box Numbering for 33 Categories of Support (Ref. A9)

TABLE A22

Support Measures for Rock Masses of "Extremely Poor" and "Exceptionally Poor" Quality (Q Range: 0.1-0.001)

Support Category	Q	Conditional Factors RQD/J_n	J_r/J_a	SPAN/ESR(m)	P Kg/cm^2 (approx.)	SPAN/ESR (m)	Type of Support	Note: see Ref. A1
33*	0.1-0.01	$\geqq$ 2	---	---	6	1.0-3.9	B (tg) 1 m + S (mr) 2.5-5 cm	IX
		< 2	---	---			S (mr) 5-10 cm	IX
		---	---	---			S (mr) 7.5-15 cm	VIII, X
34	0.1-0.01	$\geqq$ 2	$\geqq$ 0.25	---	6	2.0-11	B (tg) 1 m + S (mr) 5-7.5 cm	IX
		< 2	$\geqq$ 0.25	---			S (mr) 7.5-15 cm	IX
		---	< 0.25	---			S (mr) 15-25 cm	IX
		---	---	---			CCA (sr) 20-60 cm + B (tg) 1 m	VIII, X, XI
35 See Note XII	0.1-0.01	---	---	$\geqq$ 15 m	6	6.5-28	B (tg) 1 m + S (mr) 30-100 cm	II, IX, XI
		---	---	$\geqq$ 15 m			CCA (sr) 60-200 cm + B (tg) 1 m	VIII, X, XI, II
		---	---	< 15 m			B (tg) 1 m + S (mr) 20-75 cm	IX, XI, III
		---	---	< 15 m			CCA (sr) 40-150 cm + B (tg) 1 m	VIII, X, XI, III
36*	0.01-0.001	---	---	---	12	1.0-2.0	S (mr) 10-20 cm	IX
		---	---	---			S (mr) 10-20 cm + B (tg) 0.5-1.0 m	VIII, X, XI
37	0.01-0.001	---	---	---	12	1.0-6.5	S (mr) 20-60 cm	IX
		---	---	---			S (mr) 20-60 cm + B (tg) 0.5-10. m	VIII, X, XI
38 See	0.01-0.001	---	---	$\geqq$ 10 m	12	4.0-20	CCA (sr) 100-300 cm	IX
		---	---	$\geqq$ 10 m			CCA (sr) 100-300 cm + B (tg) 1 m	VIII, X, II, XI
		---	---	< 10 m			S (mr) 70-200 cm	IX
		---	---	< 10 m			S (mr) 70-200 cm + B (tg) 1 m	VIII, X, III, XI

*Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

EXAMPLE OF CALCULATIONS
20 m SPAN MACHINE HALL IN PHYLLITE

1. Rock Mass Classification

Joint Set 1 — Strongly developed foliation

Smooth-planar ($J_r = 1.0$)

Chlorite coatings ($J_a = 4.0$)

Ca. 15 joints/metre

Joint Set 2 — Smooth-undulating ($J_r = 2$)

Slightly altered joint walls ($J_a = 2$)

Ca. 5 joints/metre

$J_v = 15 + 5 = 20$

$J_n = 4$. Minimum $J_r/J_a = 1/4$

Minor water inflows: $J_w = 1.0$

Unconfined compression strength of phyllite (σ_c) = 400 kg/cm^2

Major principal stress (σ_1) = 30 kg/cm^2

Minor principal stress (σ_3) = 10 kg/cm^2

$(\sigma_1/\sigma_3) = 3$

$\sigma_c/\sigma_1 = 13.3$ (medium stress) SRF = 1.0

$$Q = \frac{50}{4} \cdot \frac{1}{4} \cdot \frac{1}{1} = 3.1 \text{ (poor)} \qquad \text{(A9)}$$

Type of excavation	Machine hall B = 20 m, H = 30 m (ESR - 1.0) B/ESR = 20, H/ESR = 30		
Support category	(a) Roof Q = 3.1; category 23 (b) Walls "Q" = 3.1 2.5; category 20		
Recommended	(a) Category 23	Table	B(tg) 1.4 m (Roof) + S(mr) 15 cm Notes: II, IV, VII
	(b) Category 20	Table	B(tg) 1.7 m (Walls) + S (mr) 10 cm Notes: II, IV

Mean length of bolts and anchors	(a) Roof:	bolts	5.0 m
		anchors	8.0 m
	(b) Walls:	bolts	6.5 m
		anchors	10.5 m

Support pressure estimates

(a) Roof Q = 3.1

1. (Fig. A13, shaded envelope) Approx. range for P_{roof} = 0.9 - 2.0 kg/cm^2
2. (Equ. A6) P_{roof} = 1.37 kg/cm^2
3. (Equ. A7) P_{roof} = 0.91 kg/cm^2

(b) Walls "Q" = 3.1 · 2.5

1. (Fig. A13, shaded envelope) Approx. range for P_{wall} = 0.6 - 1.4 kg/cm^2
2. (Equ. A6) P_{wall} = 1.01 kg/cm^2
3. (Equ. A7) P_{wall} = 0.67 kg/cm^2

Comments

1. Note the use of the minimum value J_r/J_a for calculating Q. The properties of the joint set having the lowest shear strength should always be used, unless the user considers the orientation is entirely favorable such that a second joint is more unfavorable to stability, despite having a higher value of J_r/J_a.

2. The choice of 1.4 m and 1.7 m spacing for roof and wall bolts from the empirical listed ranges of 1 - 1.5 m and 1 - 2 m was made in accordance with the specific value of Q, in relation to the range for the given category (i.e., Q = 1 - 4). These bolt spacings are approximate and need to be checked against required support pressures.

3. When using Tables 11, 12, 13, and 14 for wall support, the relevent span should be used when the conditional factor (SPAN/ESR) is listed. Hence, the choice of the minimum 10 cm of mesh reinforced shotcrete from a possible range of 10 - 20 cm.

4. The mean bolt and anchor lengths should be coordinated with the recommendation given under Note II (p. 229). Thus, for the roof, variable (intermeshed) bolt lengths of 3, 5, and 7 m appear reasonable, while for the wall 5, 6.5, and 8 m might be more appropriate. The recommendation for using tensioned cable anchors (Note IV) is based on current practice in most caverns of this size. The effectiveness of such widely spaced (2 - 4 m) reinforcement is perhaps open to question.

5. The range of estimates of support pressure gives room for choice. The estimates obtained from Equ. (A7) are especially dependent on the absence of additional joint sets. Should some additional random joints be discovered when access tunnels are driven into this hypothetical rock mass, both J_n and Q will be affected, and this will have a multiple effect on Equ. (A7) The value of J_n will increase to 6, Q will reduce to 2.1, and the estimate of roof support pressure would rise from 0.91 to 1.28 kg/cm^2.

DESIGN CONCEPTS USING THE SUPPORT TABLES

The application of support dimensioning can be divided into three parts: bolting, concrete lining, shotcrete lining.

1. Bolting

The support capacity of tensioned or grouted bolts is equal to the yield capacity of one bolt divided by the square of the bolt spacing:

$$P = 1/a^2 \qquad \text{(A10)}$$

where

P = support pressure capacity in kg/cm^2

a = bolt spacing in meters

Equation (A10) and the support pressure chart (Figure A13) were used in combination with the case records to provide a reasonably continuous spectrum of bolt spacings. When a range of spacings is given in Tables A18 - A21, for example 1.5 to 2.0 m, the lower limit applies to the lowest rock mass quality Q, and the upper limit to the highest rock mass quality in each category. For supplementary reinforcement, the bolt spacings could be increased, provided the total support pressure of the combined bolting and anchoring is not reduced.

Bolt and anchor lengths depend on the dimensions of the excavations. Lengths for the roof are usually related to the span, while those used in the walls are usually related to the height. The ratio of bolt length to span tends to reduce as the span increases. This trend has been demonstrated by Benson, et al (Ref. A19). The following recommendations are given as guides to be modified as conditions demand.

A19. Benson, R.P., R.J. Conlon, A.H. Merritt, P. Joli-Coeur, and D.U. Deere, 1971, "Rock Mechanics at Churchill Falls," American Society of Civil Engineers, Symposium on Underground Rock Chambers, Proceedings 407-486, Phoenix, Arizona.

Roof:	bolts	$L = 2 + 0.15\ B/ESR$
	anchors	$L = 0.40\ B/ESR$
Walls:	bolts	$L = 2 + 0.15\ H/ESR$
	anchors	$L = 0.35\ H/ESR$

where

L = length in metres

B = span in metres

H = excavation height in metres

ESR = excavation support ratio

2. Concrete Lining

The theory of thin-walled cylinders provides the relation between lining thickness, stress in lining, and uniform internal or external pressure at equilibrium. For external loading the following applies:

$$t = \frac{P \cdot R}{\sigma} \qquad \text{(A11)}$$

where

P = externally applied pressure (kg/cm^2)

σ = compressive stress in lining (kg/cm^2)

R = internal radius of lining (cm)

t = wall thickness for equilibrium (cm)

Equation (A11) is based on the assumption that bending and shear stresses are absent.

When a concrete lining is used in combination with bolting, stresses caused by uneven loading or noncircular lining are presumably minimized and equation (A11) gives a conservative value for allowable stress. If bolt tensions could be guaranteed, some sharing of support pressure would occur and lining thickness could be reduced. However, some form of

internal steel reinforcement may be required to reduce the unfavorable effect of uneven stresses. A conservative value of σ (allowable) equal to 50 kg/cm^2 was assumed for the values in Tables A18 - A19. The appropriate range of pressure (P) was estimated using Figure A13 in combination with case records.

Support pressure load sharing by systematic bolting was ignored; therefore, concrete thickness may be too conservative if bolts are used. Concrete lining is recommended for only the poorest qualities of rock mass, where the effectiveness of bolt anchorage is uncertain.

3. Shotcrete Lining

When single (2 - 3 cm) or double (5 cm) applications of shotcrete are applied, usually in combination with bolting (i.e., support categories 21 and 25, Tables A19 - A20, the function of the shotcrete is to prevent loosening in the zone between bolts. In such cases, no attempt was made to use equation (A11) for design thicknesses. The mode of failure of thin layers of shotcrete is one of shear, not bending or compression, as emphasized by Rabcewicz (Ref. A20) and Muller (Ref. A21). The values in the tables are based on a wealth of case records in these support categories, and theoretical applications are not necessary.

Cost Considerations

Equipment Costs. The cost of construction of a tunnel consists of a combination of fixed and variable costs. Fixed costs depend upon the

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A20. Rabcewicz, L.V., 1969, "Stability of Tunnels Under Rock Load," Water Power, Vol. 21, June: 225-229, July: 266-273, August: 297-302.

A21. Muller-Salzburg, L., 1970, "A New European Tunneling Concept," paper presented at a Tunneling Conference at Lorch, West Germany under the title "Neuere Auffassungen im mitteleuropaischen Felshohlraumbau und deren Auswirkungen auf die Praxis," Salzburg, p. 42, Osterreichische Gesellschaft fur Geomechanik, Translation 17.

design of the tunnel, the location, grade, alignment, type of lining, cost of mobilization of special equipment, such as a TBM, and the total length and type of a specific tunnel in a specific location. Fixed costs, as related to materials, refer to those that the contractor is required to place in the ground and leave in place, such as the lining, both temporary and secondary. Lining costs may represent 30% or more of the total cost, of which some 50% consists of materials alone. The lining required depends on the local geology, but it may be significantly influenced by the function of the tunnel.

For the proposed deep-based missile system in a geologic environment consisting of sandstones and associated rocks at 2,000 - 3,000 foot depth, the lining must support the static loads imposed by the rock pressure, provide for movement of equipment, etc., and be strong enough in some areas to offer some resistance to dynamic loading. Hence, the fixed costs will include a steel reinforced concrete lining, probably placed in segments.

Variable costs are those which depend upon time, such as labor, cost of utilities, cost of supervision, rental of special equipment, and costs amortized over the life of the project. A significant cost factor is the length of time that the contractor is on site, which depends on the rate of advance of the tunnel or the number of feet driven per day.

Rates of advance may vary widely. At high rates, the saving due to increasing the rate of advance decreases and the fixed costs become more significant in determining the total cost.

Significant improvements have occurred in factors which affect rates of advance in the last few years. These consist of greater mechanization of mining methods, the development of new techniques for removing muck, for materials handling, for pumping of concrete, and the availability of various types of tunnel boring machines which have contributed to these

improvements. Developments in the components of boring machines is the most significant.

The construction of better cutters and bearings has markedly decreased the downtime in TBM operation. However, these types of improvements are approaching optimistic levels, and further improvements must come from other types of design, development of machine parts, and methods of operation.

The following are important factors which will have a decisive effect upon an extensive project such as the comtemplated DBM system.

The volume of tunnel construction in a given geographical area affects the costs of construction of tunnels. For smaller projects, if there is a variety of tunneling equipment available in a local area, costs will decrease.

A critical factor of major significance in large projects is the availability of experienced tunneling crews. Tunnel construction requires specific skills, and labor experienced in building construction or other types of earthwork construction cannot be readily and efficiently retrained as tunnel crews. The interdependence and coordination of individual workmen in a tunnel is more significant than in other construction projects. Because of these and other labor requirements, tunnel contractors are specialists in their area of operations and depend on tunneling construction for the major proportion of their revenue.

The interdependence of these and related factors is extremely complex, and it is generally not possible to identify exactly the economic significance of any one factor in overall tunnel costs.

Mayo, et al (Ref. A22), from an extensive study of tunneling projects, concluded that rates of advance in hard rock tunneling could be increased

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A22. Mayo, Robert S., Adair Thomas, and Robert J. Jenny, January 1968, "Tunneling - The State of the Art," Robert S. Mayo & Associates, published for U.S. Dept. of Housing and Urban Development, Washington, D.C., USA.

to 70 to 90 ft/day. These increases in advance rates must include faster muck removal, more rapid installation of support systems, and more efficient operations in general. Costs must be continually updated to provide for escalation of labor, material, and related cost factors.

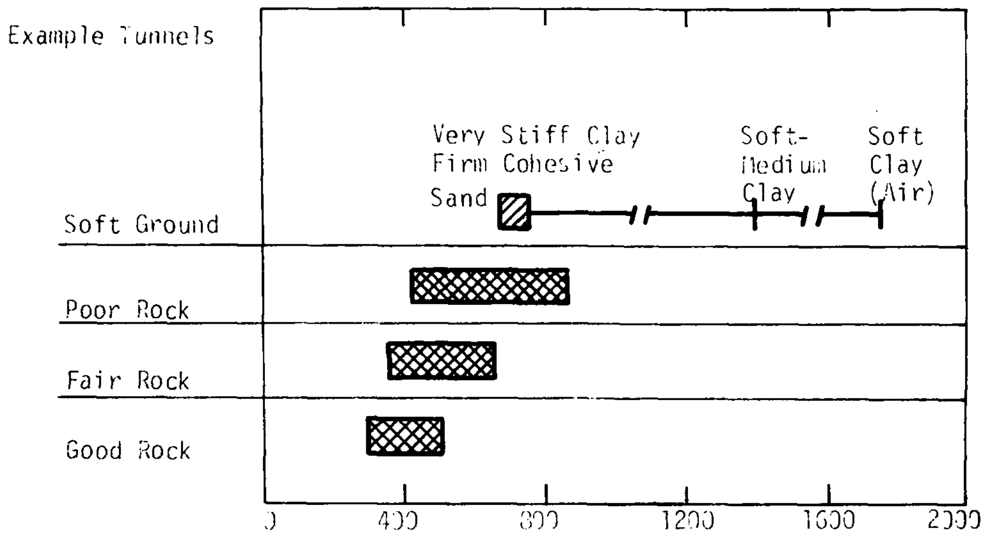
Peck, et al (Ref. A23), made an extensive study of the design of supports for tunnels in both soft ground and rock for an 18-ft diameter "example tunnel" with a variety of ground conditions and support systems. A synthesis of costs was prepared and compared to available data on costs for other tunnels reported in the literature (Figure A16). The price per foot for tunnels in soft ground overlaps the high range of cost for steel ribs in poor rock. In general, the cost of tunnels in soft ground is close to that for tunnels in poor rock using steel rib support.

Significant conclusions of this report were:

1. Tunneling must be treated as a complete system, not as a collection of unrelated components. The total cost of tunnel construction can be reduced either by reducing the cost of one or more of the components without unduly affecting the rate of advance or by increasing the rate of advance without excessively raising the costs of any of the components.

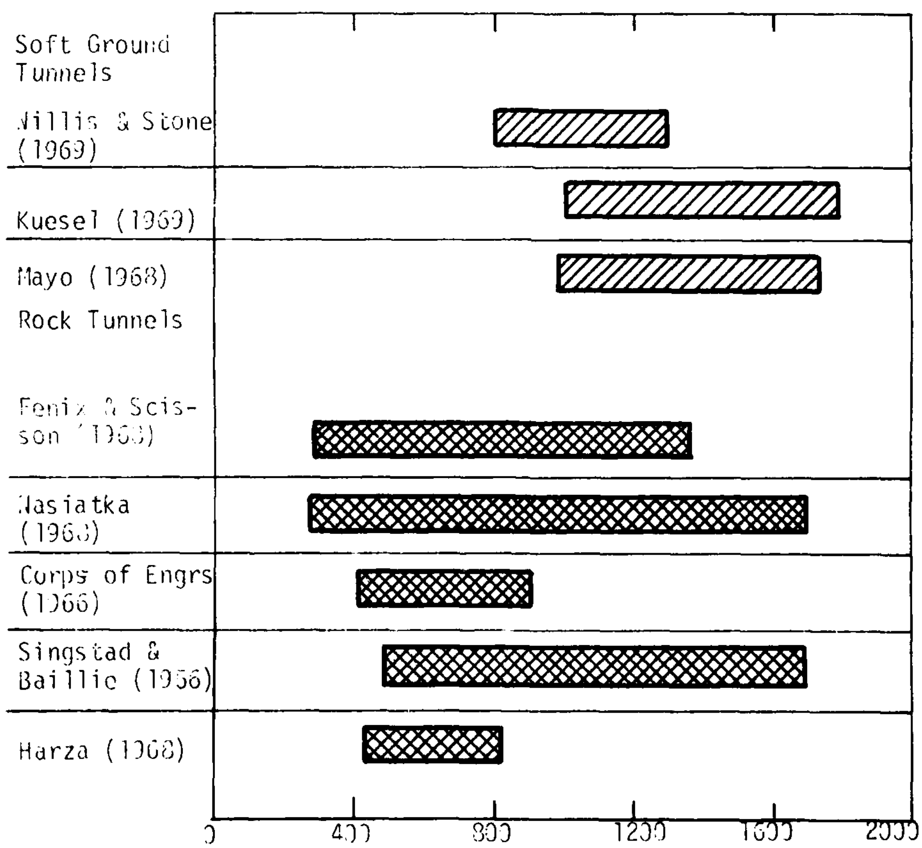
2. The percentage saving possible by reductions in new applications of primary support may be less than 10% of the total cost of the tunnel; however, major savings, perhaps as high as 30% of the total cost of the tunnel, might be achieved by designing primary support systems that do not require secondary linings of concrete.

A23. Peck, R.B., D.U. Deere, J.E. Monsees, H.W. Parker, and B. Schmidt, 1909, "Some Design Considerations in the Selection of Underground Support Systems," Report for U S. Dept. of Transportation, OHGST, Contract 3-0152. Published by the National Technical Information Service, Springfield, Virginia, USA.



Total Cost of Tunnel-Dollars/Ft

Range of Cost - Example Tunnels



Total Cost of Tunnel-Dollars/Ft
Range of Cost - Data Reported in Literature
*Normal Range is Shown Extreme Range is $225/ft to $3600/ft

FIGURE A16 - Cost Comparison of Soft Ground and
Rock Tunnels (Ref. A26)

Spittel, et al (Ref. A24), in a review and analysis of tunnel construction costs for the Office of High Speed Ground Transportation used data for cost analyses from historical records of contractors, owners, and manufacturers throughout the United States. Unfortunately, detailed data or actual project performance as measured by rates of advance were unavailable for most of the projects examined. The general conclusions in this report are pertinent:

1. For tunnel diameters (up to 40 ft), there is a marked increase in costs with increasing diameter. However, in the range of 10- to 15-ft diameters, this is relatively modest.

2. The trend of increase in tunneling costs to 1971 is much less than that for other segments of the construction industry. Increased use of machine tunneling has played a major role in restraining escalating costs.

3. To derive the greatest benefit from a tunneling machine, the contractor must strive for the most intensive use of this equipment. However, tunneling machines are presently being designed for each specific tunnel project and only rarely, if ever, are they not written off on the job for which they were purchased. Costs could be definitely reduced if such machines could be used for several projects.

4. Coincident with 2, tunnels should be standardized with about 6 to 8 sizes and one shape. However, standardization of the tunnel diameter and shape without a commensurate criterion for lining thickness would only effect modest improvements in costs.

A24. Spittel, Louis A. and J.C. Willard, March 1951, "Tunneling Cost Analysis," RMC, Inc., Bethesda, Maryland, U.S. Dept. of Transportation.

5. The criteria for contract awards and construction specifications could, in many cases, be changed to produce a lower final overall cost of construction.

An index for tunneling costs in Toronto (Figure A17) was taken from data published in the Engineering News record. The cost items not directly related to tunneling were excluded, such as shafts, manholes, and appurtenant work. The total cost for the project was divided by the tunnel length to give the cost per foot. These adjusted costs (Figure A17) show a rapid increase.

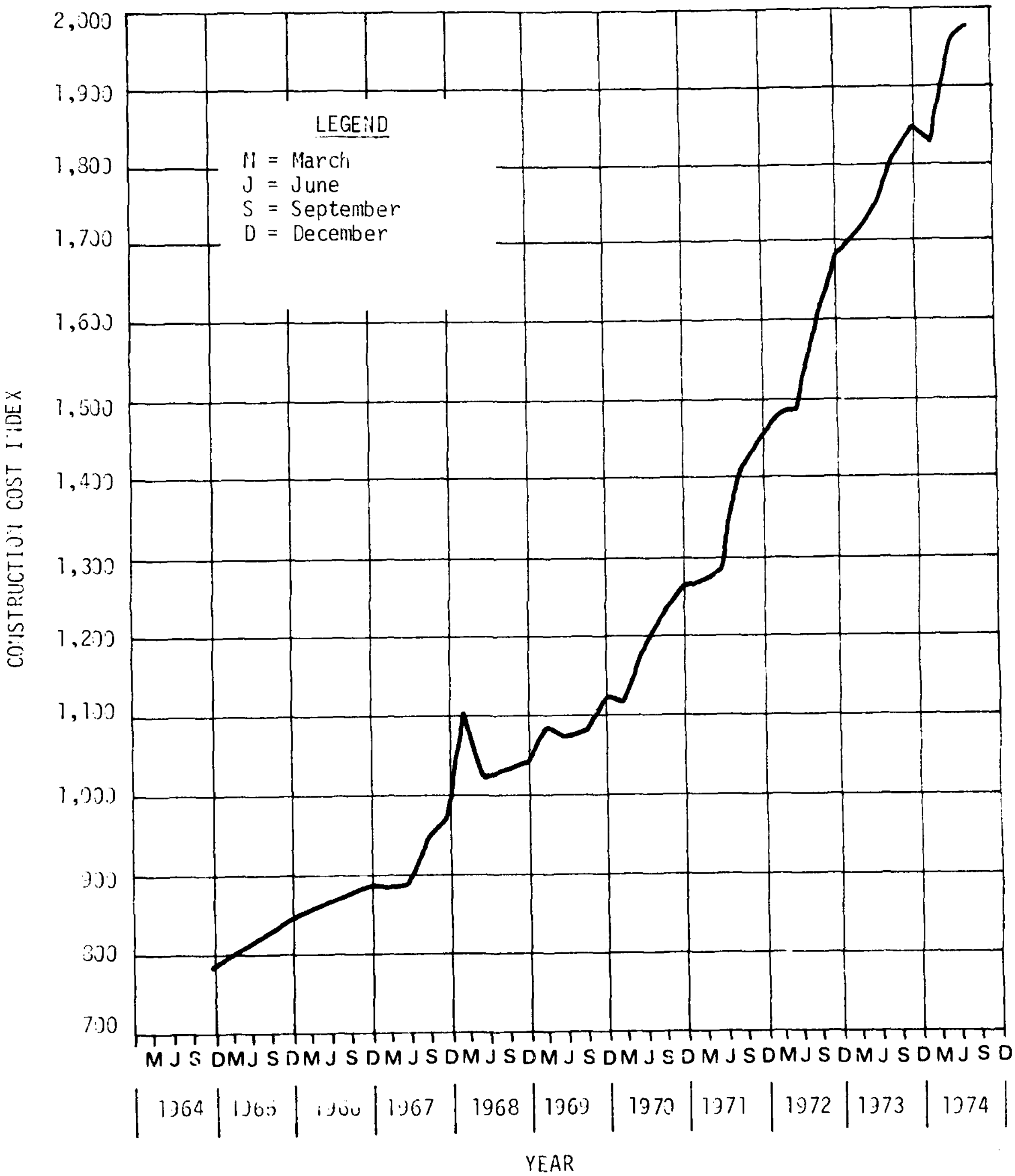


FIGURE A17 - ENR (Toronto) Construction Cost Index

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